UC/CLC CAMPUS EARTHQUAKE PROGRAM

Strong Earthquake Motion Estimates for Three Sites on the U. C. San Diego Campus

S. Day, SDSU
M. Doroudian, UCLA
A. Elgamal, UCSD
S. Gonzalez, SDSU
F. Heuze, LLNL
T. Lai, UCSD
B. Minster, UCSD
D. Oglesby, UCR
M. Riemer, UCB
F. Vernon, UCSD
M. Vucetic, UCLA
J. Wagoner, LLNL
Z. Yang, UCSD

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PREFACE

This report was prepared under the UC/CLC Campus Earthquake Program. The project was initiated as part of the Campus-Laboratory Collaboration (CLC) Program created by the University of California Office of the President (UCOP).

The UC/CLC Campus Earthquake Program (CEP) started in March 1996, and has involved a partnership among seven campuses of the University of California - Berkeley, Davis, Los Angeles, Riverside, San Diego, Santa Barbara, Santa Cruz - and the Lawrence Livermore National Laboratory (LLNL). It is designed to estimate the effects of large earthquakes on three of those campuses. Each campus has selected one to three sites to demonstrate the methods and procedures used by the CEP: Rivera Library, and parking lots (PL) 13 and 16 at U.C. Riverside, Thornton Hospital, the Cancer Center, and PL 601 at U.C. San Diego, and Engineering 1 building at U.C. Santa Barbara.

The project focuses on the estimation of strong ground motions at each selected site. These estimates are obtained by using an integrated geological, seismological, geophysical, and geotechnical approach, bringing together the unique capabilities of the campus and laboratory personnel. This project is also designed to maximize student participation. Many of the site-specific results are also applicable to risk evaluation of other sites on the respective campuses. In the future, we plan to extend the integrated studies of strong ground motion effects to other interested U.C. campuses, which are potentially at risk from large earthquakes.

To put things in perspective, the aim of the CEP is to provide University campuses with site-specific assessments of their strong earthquake motion exposure, in addition to estimates they obtain from consultants according to the state-of-the-practice, i.e. Building Codes (UBC 97, IBC 2000), and Probabilistic Seismic Hazard Analysis (PSHA). The Building Codes are highly simplified tools, while the more sophisticated PSHA is still somewhat generic in its approach because it usually draws from many earthquakes not necessarily related to the faults threatening the site under study. Eventually, both the results from the state-of-the-practice and from the CEP should be analyzed, to arrive at decisions concerning the design-basis assumptions for buildings on U.C. campuses.

This report describes how the strong ground motion estimates were obtained at U.C. San Diego, where a new seismic station was installed in 1997. The Principal Investigators at San Diego are Professors Bernard Minster and Ahmed Elgamal.

This UC/CLC project is funded from several additional sources, which leverage the core support provided by the Office of the President and which are gratefully acknowledged. These include the University Relations Program at LLNL, formerly directed by Dr. Claire Max and presently by Dr. Harry Radousky, and the offices of the appropriate Vice-Chancellors on the various campuses. At U.C. San Diego, the Vice-Chancellor Resource Management and Planning is John A. Woods, and the Campus Architect is M. Boone Hellmann.

The Director of the UC/CLC Campus Earthquake Program is Dr. Francois Heuze from LLNL.

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EXECUTIVE SUMMARY

This is the second report on the UC/CLC Campus Earthquake Program (CEP), concerning the estimation of exposure of the U.C. San Diego campus to strong earthquake motions (Phase 2 study). The first report (Phase 1), dated July 1999, covered the following topics:

- seismotectonic study of the San Diego region
- definition of causative faults threatening the UCSD campus
- geophysical and geotechnical characterization of the Thornton hospital site
- installation of the new CEP seismic station
- and, initial acquisition of earthquake data on campus.

The main results of Phase 1 are summarized in the current report.

This document describes the studies which resulted in site-specific strong motion estimates for three sites on campus: Thornton hospital, the new Cancer Center site, and Parking Lot/PL 601 site of the future Garamendi Medical Research building. The main elements of Phase 2 are:

- Concluding that a M 6.9 earthquake involving the 60-km long combined Mission Bay and Del Mar segments of the Rose Canyon fault is the main seismic hazard for the campus. Its recurrence interval is of the order of 1,800 years. The nearest distance from the surface trace of the fault segments to campus locations is 3 to 4 km.
- Creating strong motion estimates (seismic syntheses) at depth of 91 m under the Thornton hospital site, and 42 m under the PL 601 site; 300 such simulations were performed, each with the same seismic moment, but giving a broad range of motions which were analyzed for their mean and standard deviation.
- Laboratory testing, at U.C. Berkeley and U.C. Los Angeles, of soil samples obtained from drilling at the UCSD station site (Thornton hospital), to determine their response to earthquake-type loading.
- Performing nonlinear soil dynamic calculations, using the soil properties determined in-situ and in the laboratory, to calculate the surface strong motions resulting from the seismic syntheses at depth.
- Comparing these CEP-generated strong motion estimates to acceleration spectra based on the application of state-of-the-practice methods - the IBC 2000 code, the UBC 97 code, and Probabilistic Seismic Hazard Analysis (PSHA). This comparison can be used to formulate design-basis spectra for future buildings and retrofits at UCSD.

Because of the new, site-specific approach that the CEP studies represent, an extensive effort of validation is documented:

- validation of the Green's functions methodology used in the seismic syntheses of strong motions at depth
- cross-comparison of the nonlinear soil models used to obtain strong motions at the surface

The ever-growing database of strong earthquake records clearly demonstrates the potential for great variability of ground motions from site to site in a given earthquake. These variations are only reflected in a coarse way in the state-of-the-practice Probabilistic Seismic Hazard Analyses, which are rather generic. Furthermore, these variations are not adequately described by the simplified design spectra of the Building codes (UBC 97, IBC 2000). These shortcomings provide a strong justification for augmenting the state-of-the-practice estimates with site-specific studies such as done by the Campus Earthquake Program.

At UCSD, the Phase 2 studies lead to the following important conclusions:

- The motions estimated at two sites on either side of Interstate 5 (Thornton hospital and PL 601) are generally comparable. These motions are expected to be representative of those that could be expected at other campus sites where the soil profile is comparable.
- However, prior to assuming the same motions for the site of a new construction project or retrofit, it is advisable to determine the shear- and compressional-wave velocities in the top 40 meters or so of depth in order to verify the similarity with the sites studied under the CEP. The preferred method for these measurements is the suspension logging described in this report. This type of investigation would add only a modest cost to standard geotechnical investigations.
- When comparing the CEP estimates to those from the state-of-the-practice, the UBC and IBC code-based spectra accommodate a large portion of the exposure defined by the CEP study. Also, the mean CEP is between the 475 and 950-year event PSHA spectra, and the + 1 sigma CEP is between the 950 and 2375-year event PSHA spectra.
- When comparing the CEP estimates to the design basis earthquakes used for Thornton hospital, that comparison shows that the maximum credible event is well in excess of the + 1 sigma CEP spectrum and thus covers a very large part of all the scenarios considered in the CEP study. The maximum probable event, on the other hand, is substantially underestimating the range of estimated CEP motions. In consequence, the seismic design basis assumptions for UCSD should be re-examined in consultation with the campus geotechnical and structural consultants.
- A comparison of the CEP-estimated acceleration spectra and acceleration time-histories to those from recent strike-slip events in California, of comparable magnitude to that assumed for the Rose Canyon fault, indicates that the CEP motions are not overestimates.

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INTRODUCTION

1.1 The Campus Earthquake Program (CEP)

The approach of the CEP is to combine the substantial expertise that exists within the UC system in geology, seismology, and geotechnical engineering, to estimate the earthquake strong motion exposure of UC facilities. These estimates draw upon recent advances in hazard assessment, seismic wave propagation modeling in rocks and soils, and dynamic soil testing. The UC campuses currently chosen for application of our integrated methodology are Riverside, San Diego, and Santa Barbara. The procedure starts with the identification of possible earthquake sources in the region and the determination of the most critical fault(s) related to earthquake exposure of the campus. Combined geological, geophysical, and geotechnical studies are then conducted to characterize each campus with specific focus on the location of particular target buildings of special interest to the campus administrators. We drill, sample, and geophysically log deep boreholes next to the target structure, to provide direct in-situ measurements of subsurface material properties, and to install uphole and downhole 3-component seismic sensors capable of recording both weak and strong motions. The boreholes provide access below the soil layers, to deeper materials that have relatively high seismic shear-wave velocities. Analyses of conjugate downhole and uphole records provide a basis for optimizing the representation of the low-strain response of the sites. Earthquake rupture scenarios of identified causative faults are combined with the earthquake records and with nonlinear soil models to provide site-specific estimates of strong motions at the selected target locations. The predicted ground motions are shared with the UC consultants, so that they can be used as input to the dynamic analysis of the buildings.

Thus, for each campus targeted by the CEP project, the strong motion studies consist of two phases, Phase 1 – initial source and site characterization, drilling, geophysical logging, installation of the seismic station (Figure 1.1), and initial seismic monitoring, and Phase 2 – extended seismic monitoring, dynamic soil testing, calculation of estimated site-specific earthquake strong motions at depth and at the surface, and , where applicable, estimation of the response of selected buildings to the CEP-estimated strong motions.

1.2 Previous CEP Studies Completed at U.C. San Diego

The Phase 1 studies were completed in 1999, and are reported in detail in Minster et al, 1999. The main results are summarized below.

An extensive review of the seismotectonics of the San Diego region was completed. It drew heavily from the work of the Southern California Earthquake Center (Jackson et al, 1995) and from the analysis of Anderson et al (1989). On the basis of this review, we judged that the most severe seismic exposure is from the Rose Canyon fault, at a distance of 3 to 4 km from the campus. The maximum earthquake magnitude is estimated at M 6.9, for a 60-km long combined rupture of the Mission Bay and Del Mar segments (Anderson et al, 1989).

The seismic, geophysical and geotechnical site characterization included the following tasks:

- P and S-wave seismic refraction surveys at several campus locations
- four cone-penetration tests (CPT) to depths up to 35 m, at sites surrounding the Thornton hospital. These tests included shear-wave velocity measurements.
- P and S-wave suspension logging of the 92-m deep hole at the location of the new seismic station.

The seismic velocity profile of the soil column at Thornton is shown in Figure 1.2. The increase in the P-wave velocity without a corresponding increase in the S-wave speed shows a perched water table between 33 and 43m depth.

The new UCSD seismic station was installed in 1997. The station has three 3-component seismometers at 46 and 92m depth and at the surface. Details of the station configuration are given in Minster et al, 1999. The data are sent to a Sun Sparcstation in Thornton which, in turn, sends them over an Ethernet link to IGPP at Scripps using the already-existing UCSD Ethernet campus-wide network. Once at IGPP, the data are archived and associated with existing catalogs in real time, and the system allows rapid access to those via the Internet.

Figure 1.1 The new CEP seismic station at UCSD's Thornton hospital

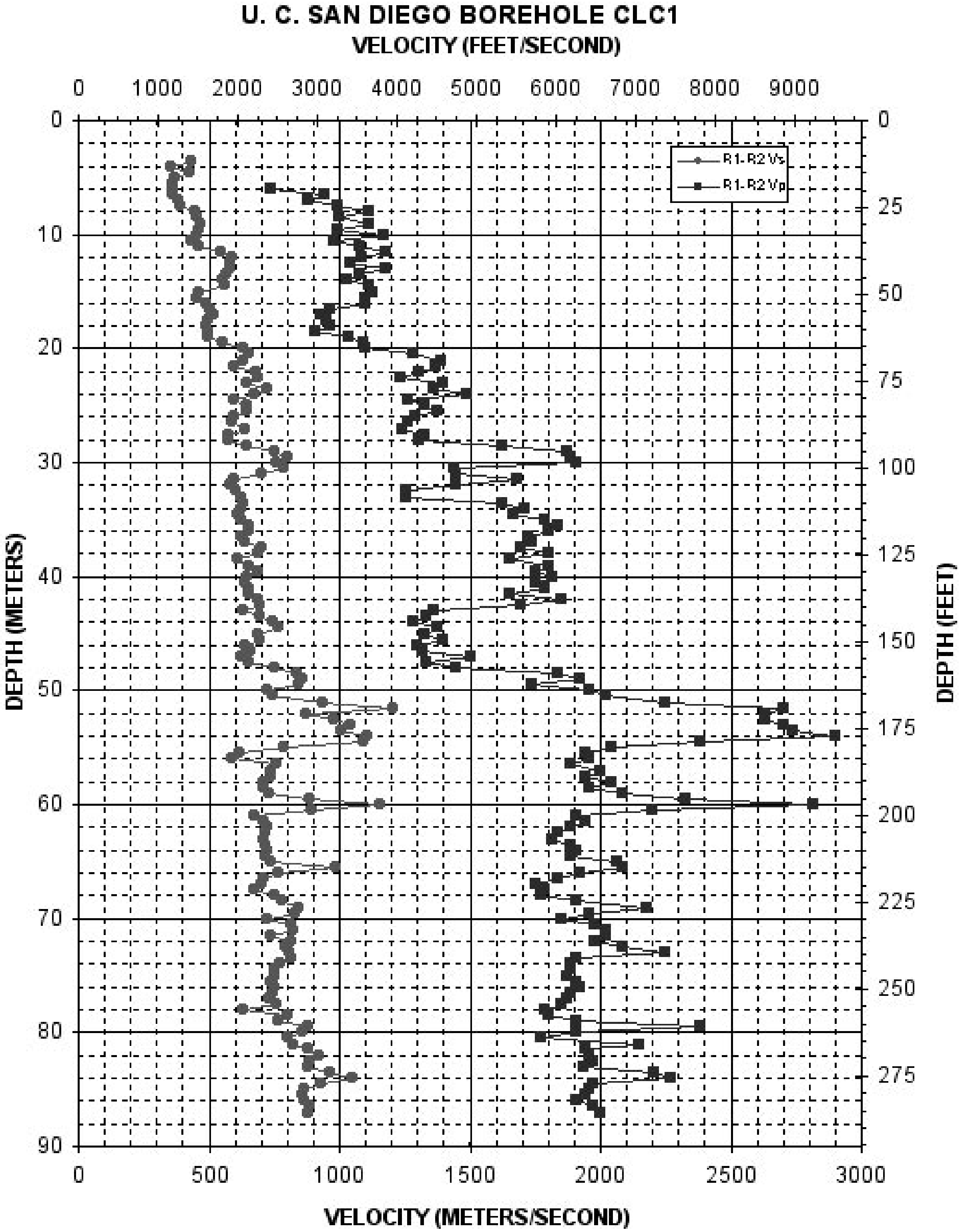


Figure 1.2. Seismic velocity profile of the Thornton hospital site.

2.0 NEW SEISMOLOGICAL STUDIES

2.1 New Earthquake Records

Numerous earthquakes have been recorded at the new Thornton hospital seismic station since its installation. Some of these events were listed in the UCSD Phase 1 report (Minster et al., 1999). However, unlike at U.C Riverside and U.C. Santa Barbara, no earthquake was recorded on the fault considered to be the main threat to the campus, in this case the Rose Canyon fault. Thus, as discussed below, the methodology for creating strong motion syntheses for UCSD was somewhat different than the one used for the other two campuses.

2.2 Down-Hole Strong Motion Syntheses

2.2.1 Method

The basic principle used in the simulations is the representation theorem (Aki and Richards, 1980). This theorem states that the ground motion observed at a location is the spatial integral over the fault plane of the temporal convolution of the source time-function with a Green's function. The source time-function may vary from point to point on the fault as may the Green's function. This is the basic method used in kinematic modeling of seismic sources. The key ingredients in this method are the specifications for the source time-function and for the Green's function. Because of the absence of earthquake records on the Rose Canyon fault, we have used a theoretical Green function approach, which substitutes theoretically-calculated Green function seismograms for the small earthquake recordings used in the empirical Green function (EGF) method.

2.2.2 Validation

The validation of the EGF method was presented in details in the UCSB (Archuleta et al, 2000a) and UCR (Archuleta et al, 2000b) Phase 2 reports. It will not be repeated here. In this study we used the method of Archuleta et al (2000a), but with the substitution of theoretical Green's functions for the EGF's. The theoretical Green function method has been validated in a number of earthquake models by different researchers. A particular example, using a method quite similar to the current one, is given in Oglesby and Archuleta (1997). In that study, the authors used theoretical Green's functions and a layered velocity structure to model the ground motion of the 1992 Petrolia earthquake in Northern California. A typical comparison between recorded and calculated waveforms is shown in Figure 2.1

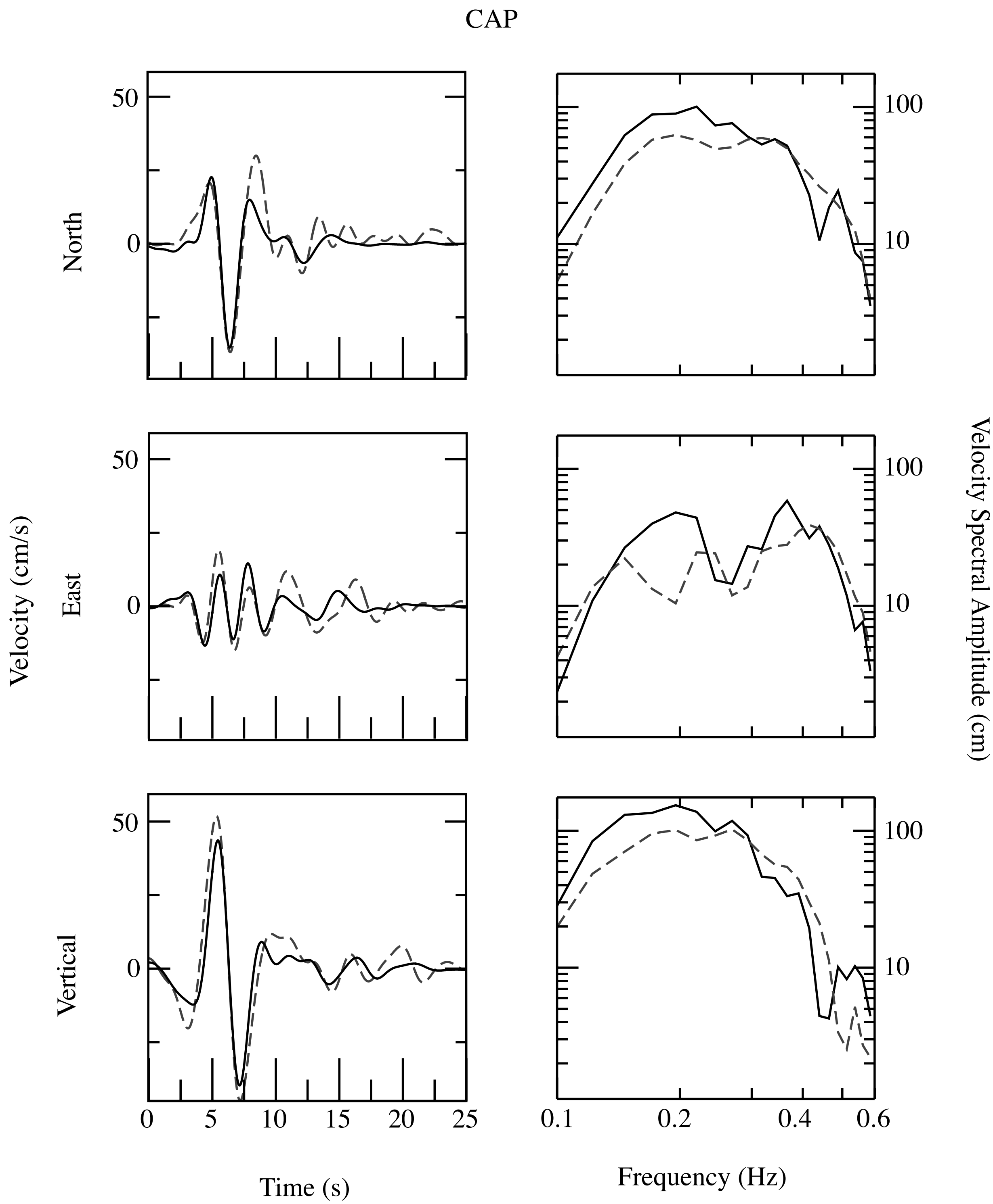


Figure 2.1 Comparison between modeled (dashed) and recorded (solid) velocities at the Cape Mendocino station for the 1992 Petrolia earthquake. The faulting model uses the theoretical Green's function method, and serves as a validation for this method (after Oglesby and Archuleta, 1999).

2.2.3 Fault Rupture Scenarios for the Rose Canyon Fault

The event modeled for the CEP estimates is a M 6.9 earthquake on the combined Del Mar and Mission Bay segments of the Rose Canyon fault (Anderson et al, 1989). The fault model is shown in Figure 2.2.

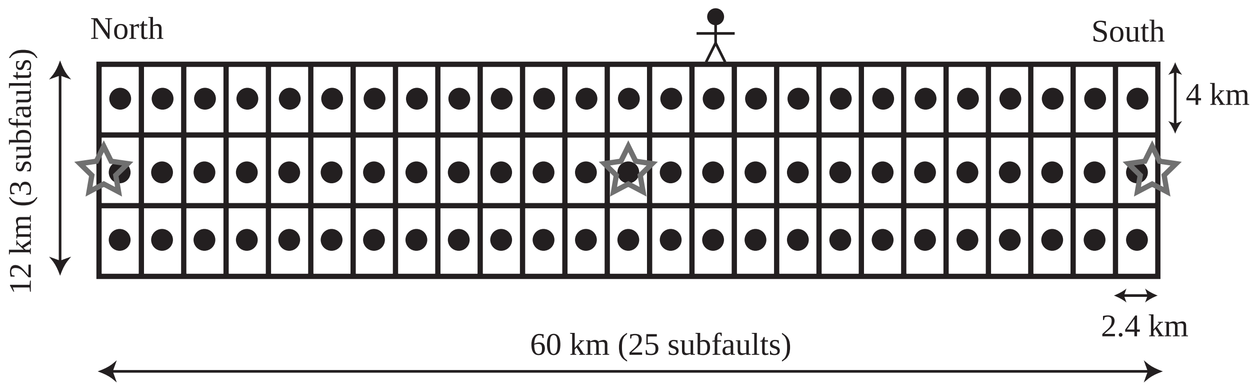


Figure 2.2. Sketch of the Rose Canyon fault model for rupture scenarios. The calculated Green's function locations are shown with black dots, and the hypocenter locations are shown with stars.

The fault model is planar and vertical. The shortest distance from the fault to Thornton hospital is 4.6 km. It should be noted that approximately only the southern 17 km of the fault is on land. The remainder of the fault occurs underwater. The faulting parameters are given in Table 2.1, and various relevant coordinates are given in Table 2.2.

Table 2.1: Faulting parameters for rupture scenarios on the Rose Canyon fault

Fault length	60 km
Fault width	12 km
Strike	147^o
Dip	90^o
Rake	180^o
Magnitude	6.9
Seismic moment	2.24×10^{26} dyne-cm
Corner frequency	0.09 Hz
Stress drop	3.0 MPa ((30 bars)
Number of calculated Green’s functions	75

Table 2.2: Coordinates of various fault and site locations for UCSD.

Location	Latitude	Longitude
North end of the fault	33.1657	-117.504
South end of the fault	32.7284	-117.170
Northern hypocenter	33.1748	-117.510
Central hypocenter	32.9470	-117.337
Southern hypocenter	32.7193	-117.163
Thornton hospital	32.8788	-117.227

We calculated theoretical Green's functions for the subfaults indicated in Figure 2.2 using a frequency-wavenumber integration method (Bouchon, 1981; Kennett, 1983). All synthetics were calculated for a receiver at the Thornton hospital site. The result is the complete linear elastodynamic response of the Thornton site to a step function in slip (i.e., an impulse in slip rate). In an elastic whole space, the far-field response is simply an impulse in displacement. In the near field on the surface of a layered medium, the Green's functions are somewhat more complicated but still have an essentially infinite corner frequency.

Using the method of Archuleta et al. (2000a), we interpolated these Green's functions across the fault (10,000 total Green's functions, or 133 interpolated Green's functions per theoretically calculated one) and summed them for different randomized slip distributions. Each function carried approximately 1/10,000 of the total moment of the earthquake. In the empirical Green's function method, the summation process includes the deconvolution of a Brune spectrum from each Green's function with a corner frequency determined empirically. This process can introduce more uncertainty into the final answer. In the theoretical Green's function method, however, since the function corner frequency is infinite, no deconvolution is necessary. The trade-off for this precision is that the theoretical Green's function method must assume a structural earth profile (which can be inaccurate), while the empirical Green's function in principle includes the true structural profile.

The structural profile used in calculating the Green's functions is given in Table 2.3. This profile is a combination of the velocity structure of Magistrale (1993) and the site-specific velocity profiles of Minster et al. (1999). The material properties are constant within layers, with discontinuities between layers. Synthetics were calculated for a receiver at the surface, using a site-related damping (kappa) term of 0.01.

Table 2.3. Velocities and densities of the Thornton site model for Green's function syntheses

Depth (m)	Vp (m/s)	Vs (m/s)	Unit weight (kg/m^3)
0	736	400	2200
20	1196	650	2248
50	1472	800	2277
60	1951	1060	2327
115	2429	1320	2377
170	2908	1580	2427
225	3386	1840	2477
280	3865	2100	2427
335	4343	2360	2577
390	4822	2620	2627
445	5300	2880	2677
500	5690	3110	2700
4000	6170	3610	2700
8000	6370	3650	2700
14000	6490	3660	2700
20000	7070	3930	2700
26000	7270	4160	2700
32000	7370	4210	2700

2.2.4 Downhole Strong Motion Estimates (Seismic Syntheses)

We calculated ground motions for 100 different randomized slip distributions and three different hypocenters on the fault (Figure 2.1), for a total of 300 scenarios. Each scenario had the same seismic moment of 2.24×10^{26} dyne-cm. All hypocenters were at a depth of 6 km. The results of these seismic syntheses were simulated strong motions calculated at the surface of the Thornton site, but neglecting soil nonlinearity. Such linear estimates of ground motions are not representative of the nonlinear soil behavior. These linear motions were then deconvolved to a depth of 91 m under Thornton hospital, using the velocity profile shown in Figure 1.2, to provide downhole incident rock motions for the Thornton/Cancer Center site. As discussed in Chapter 4, these downhole motions were then propagated upward using the nonlinear soil profile. For the Medical Building site (PL 601), the same linear motions were deconvolved to a depth of 42 m and then propagated upward up the nonlinear profile of that site. The response spectra of downhole incident motions at Thornton and the representative time-histories are shown in Figures 2.3 and 2.4. Corresponding results for PL 601 are shown in Figures 2.5 and 2.6. As discussed in Archuleta et al, (2000a) for the U.C. Santa Barbara campus motions, the + and - 1 sigma curves reflect both the parametric and the modeling (epistemic) uncertainties.

As noted by Archuleta et al., 2000a, (the report on ground motion estimates for the U.C. Santa Barbara campus), the source model we use for ground motion simulations has stochastic components. Among these, are random timing variations that are added to the slip initiation time of the model subfaults. One of the model assumptions is that the spatial correlation of these timing variations is negligible. This assumption is probably not an important limitation at short periods (less than ~ 0.5 seconds) nor at larger distances (exceeding ~10-20 km) from the fault (see the validation study in Archuleta et al, 2000a). At periods exceeding about 0.5 seconds, however, pulse coherence can lead to enhanced (reduced) near-fault ground motion due to forward (backward) rupture directivity. The directivity effect can be especially important at near-fault sites like Thornton (~ 5km from the fault). Empirically, however, that directivity effect is found to be small (less than about 20%) for periods less than 1 second, although it can become quite large at longer periods (Somerville et al., 1997). Because it was beyond the scope of the present project to model these longer periods, coherent near-fault motions, our estimates of both median response spectral ordinates and their standard deviations at periods exceeding 1 second should be considered lower bounds for these quantities.

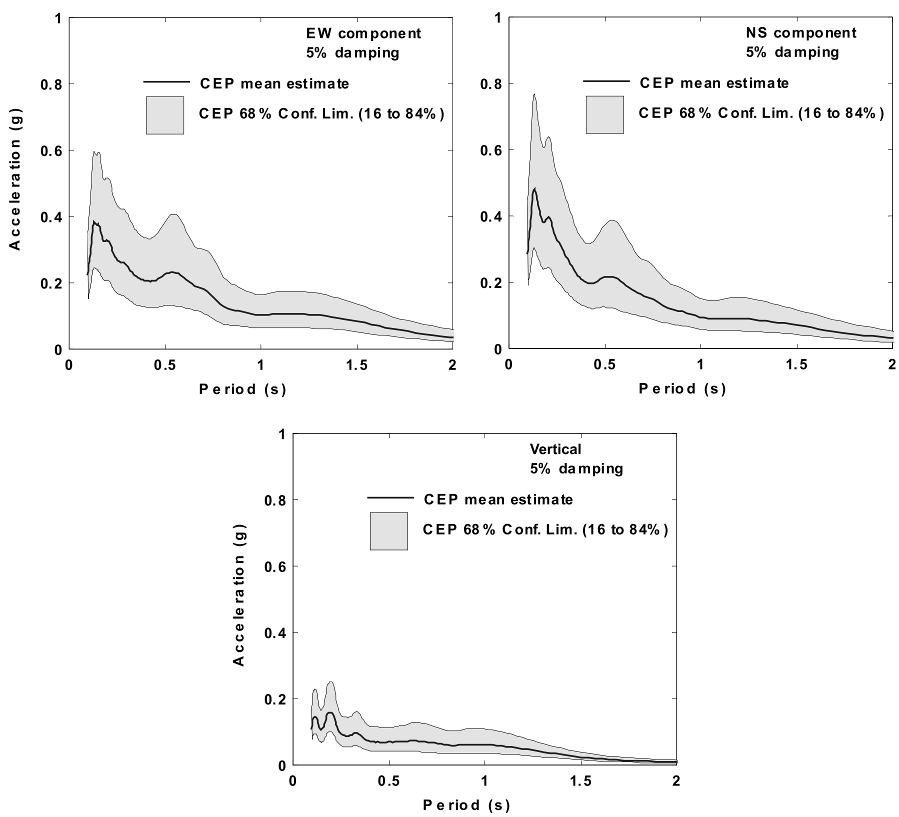


Figure 2.3 Acceleration response spectra of downhole incident motions at – 91m at the Thornton/Cancer Center site

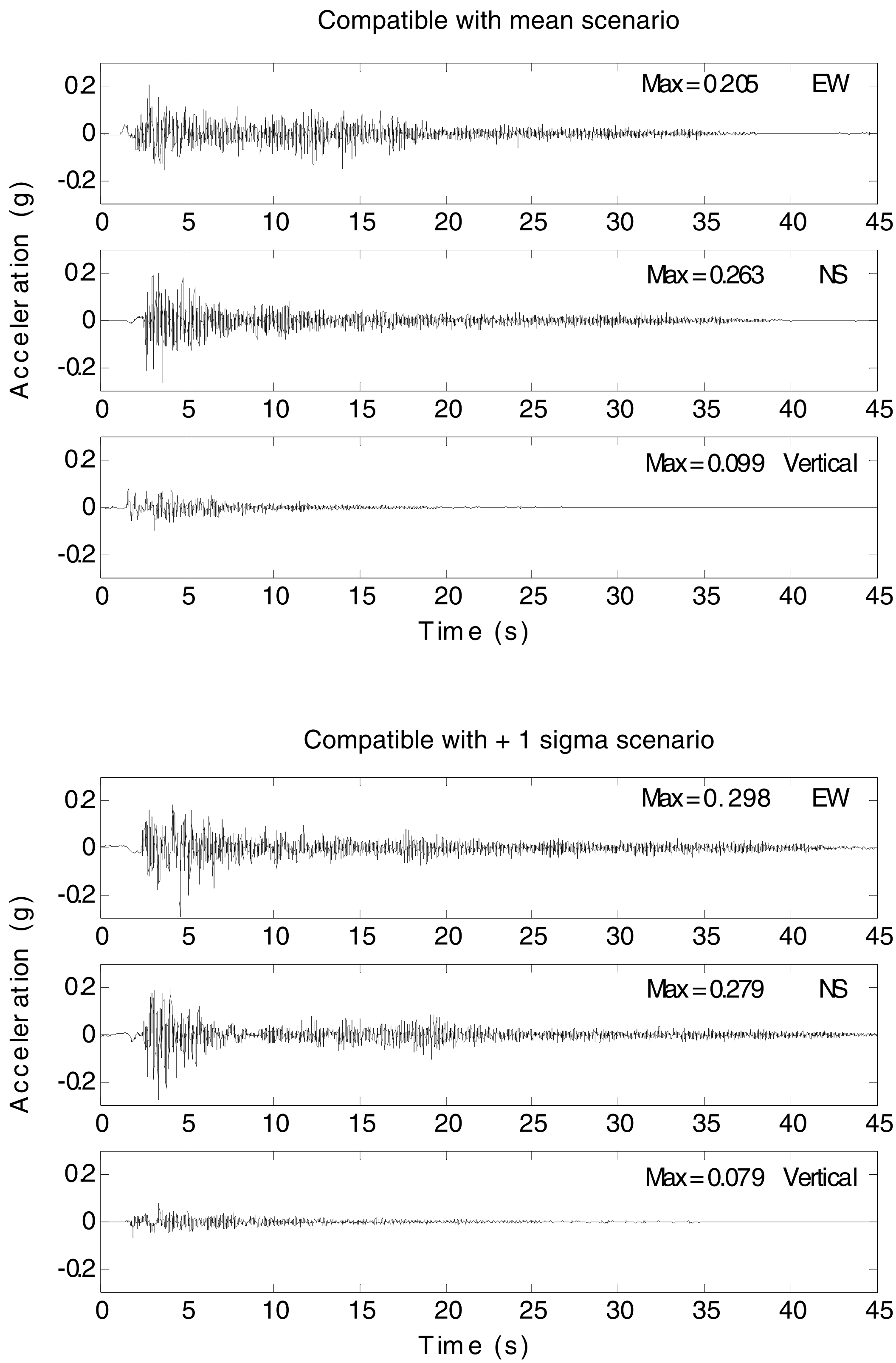


Figure 2.4 Representative acceleration time-histories for downhole incident motions at the Thornton/Cancer site

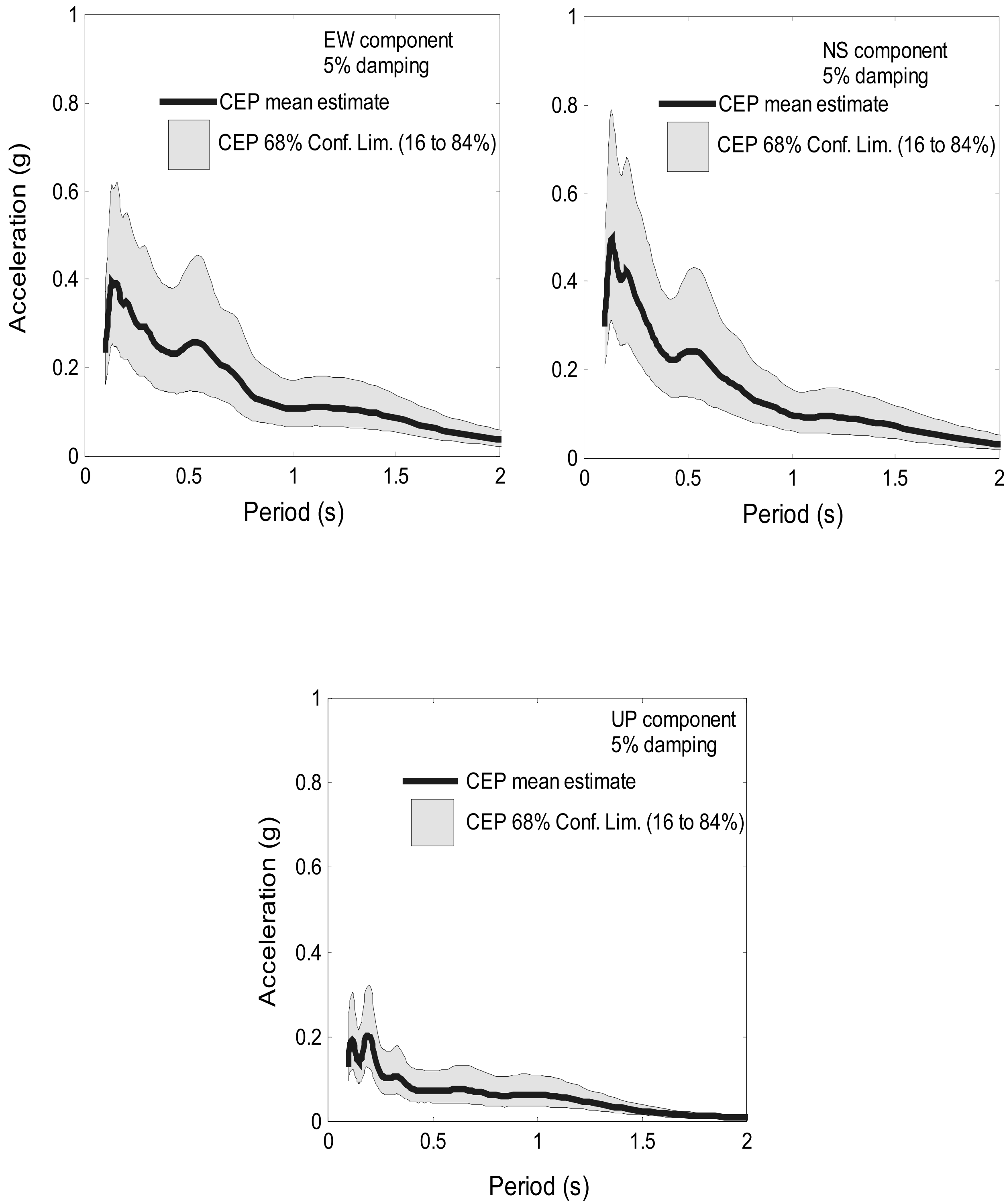
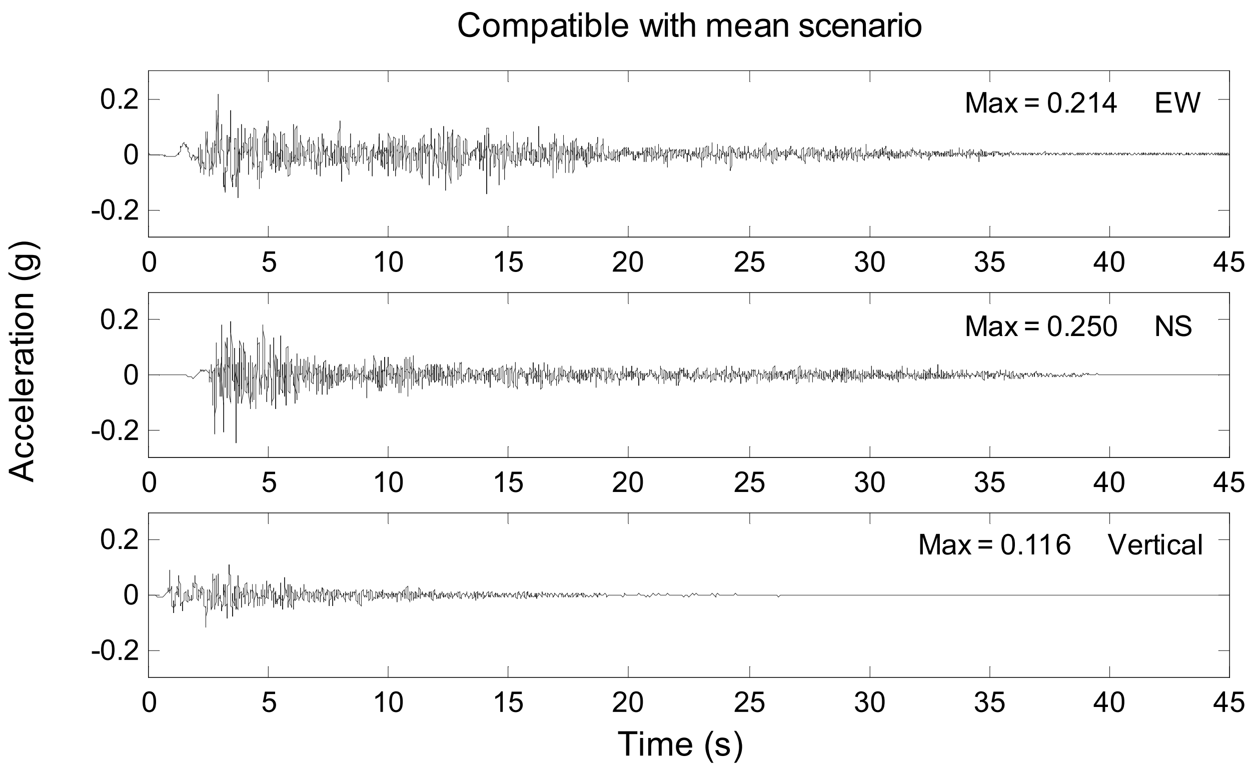


Figure 2.5 Acceleration response spectra of downhole incident motions at – 42m at the Medical Building /PL601 site



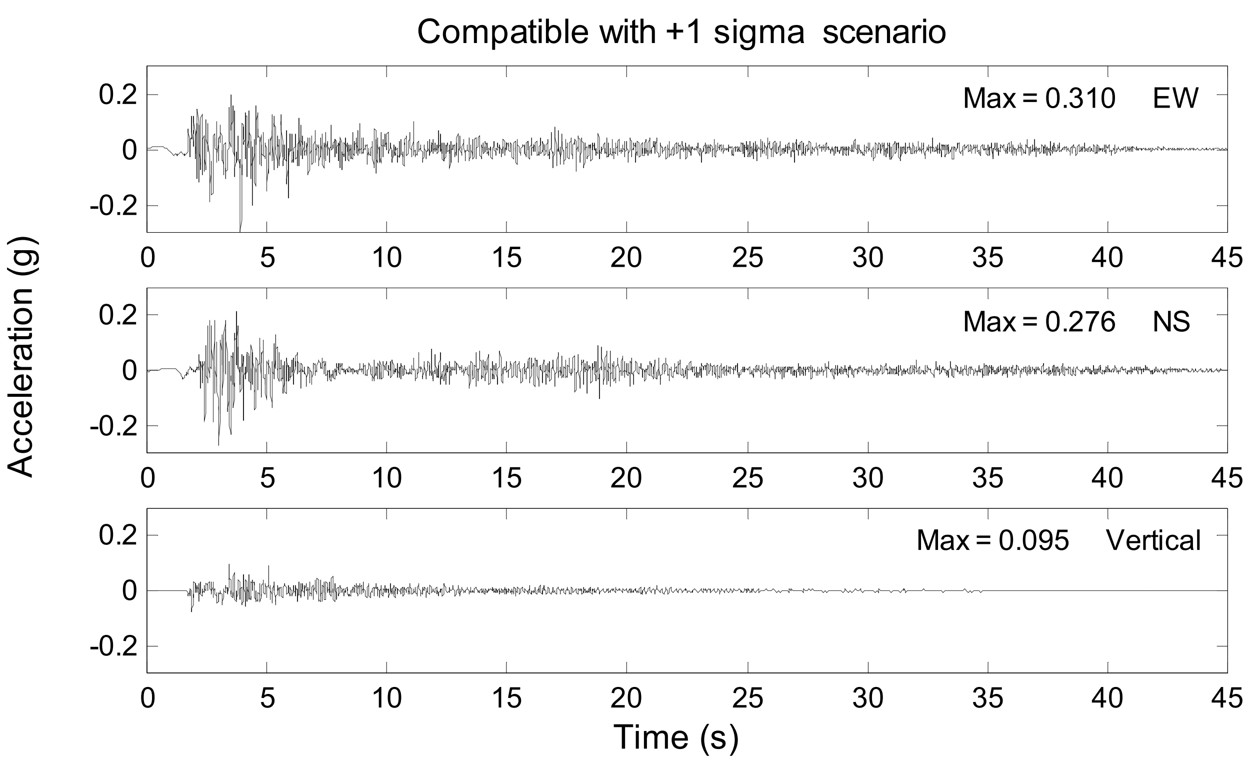


Figure 2.6 Representative acceleration time-histories for downhole incident motions at the Medical Building/PL 601 site

3.0 NEW DRILLING AND GEOPHYSICAL LOGGING

3.1 Logs of the New Holes at PL 601 and the Cancer Center

Two new holes were drilled during this phase of the project. The first one was at Parking Lot 601, site of the future Garamendi Medical Research building; the other one was at the site of the future Cancer Center southeast of Thornton hospital. Because of problems with the drilling company in terms of its ability to drill deep holes and to maintain borehole stability at both sites, the final drilling and logging depths were less than originally planned. Actual results were:

- PL 601: drilled to 160 ft (48 m); samples obtained to 55 ft (16.5 m); logged with P- and S-wave suspension tools by Geovision Inc., of Corona, CA to 152 ft (44 m); logged with gamma and resistivity tools by Welenco, of Bakersfield, CA to 160 ft (48 m).
- Cancer Center: drilled to 165 ft (49.5 m); no samples were called for since the nearby Thornton site had been sampled; logged with gamma and resistivity tools by Welenco, of Bakersfield, CA to 160 ft (48 m); logged with P- and S-wave suspension tools by Geovision Inc., of Corona, CA to 58 ft (17.5 m). The delays in drilling at this site created a delay in the P-S logging during which time the hole caved and could not be logged deeper.

The new P and S-wave velocity profiles are shown in Figures 3.1 and 3.2, and all three sites are compared in Figure 3.3. The P-logs of PL 601 show a substantial amount of perched water at different horizons; that is believed to be due to the drilling, which did not used mud and thus injected water in the formation. The formation at a depth of 45 m at PL 601 qualifies as a soft rock/rock, i.e. at the border between B and C site classes in the terminology of the IBC and UBC building codes. Accordingly, the incident downhole motions will be input at this depth and propagated up the soil column to estimate the surface strong motions at PL 601. Based on the data from the three UCSD sites, we conclude that the ground motions relevant to seismic input to building design can be considered to be effectively in dry soils.

3.2 Comparison of the Logs at the Cancer Center and Thornton Sites

The shear velocities at the Cancer Center are comparable to those at Thornton to a depth of 17.5 m. Below that depth, the comparison is based on the gamma logs (Figure 3.4). To establish a common reference frame, the data are presented as a function of elevation above Mean Sea Level (MSL). The hole collar elevations were respectively 343 ft (104.6 m) and 336 ft (102.5 m) at Thornton and the Cancer Center. There is a definite correlation between the two profiles, except in some discrete horizons. This is emphasized in Figure 3.5 where the traces have been superimposed with a small horizontal shift, for best fit.

Based on the proximity of the Cancer Center site to the Thornton site and on the fits of the shear velocity and gamma logs over the first 80 ft (24 m) of depth, we decided to adopt the same strong motion estimates for the Cancer Center as those for Thornton hospital.

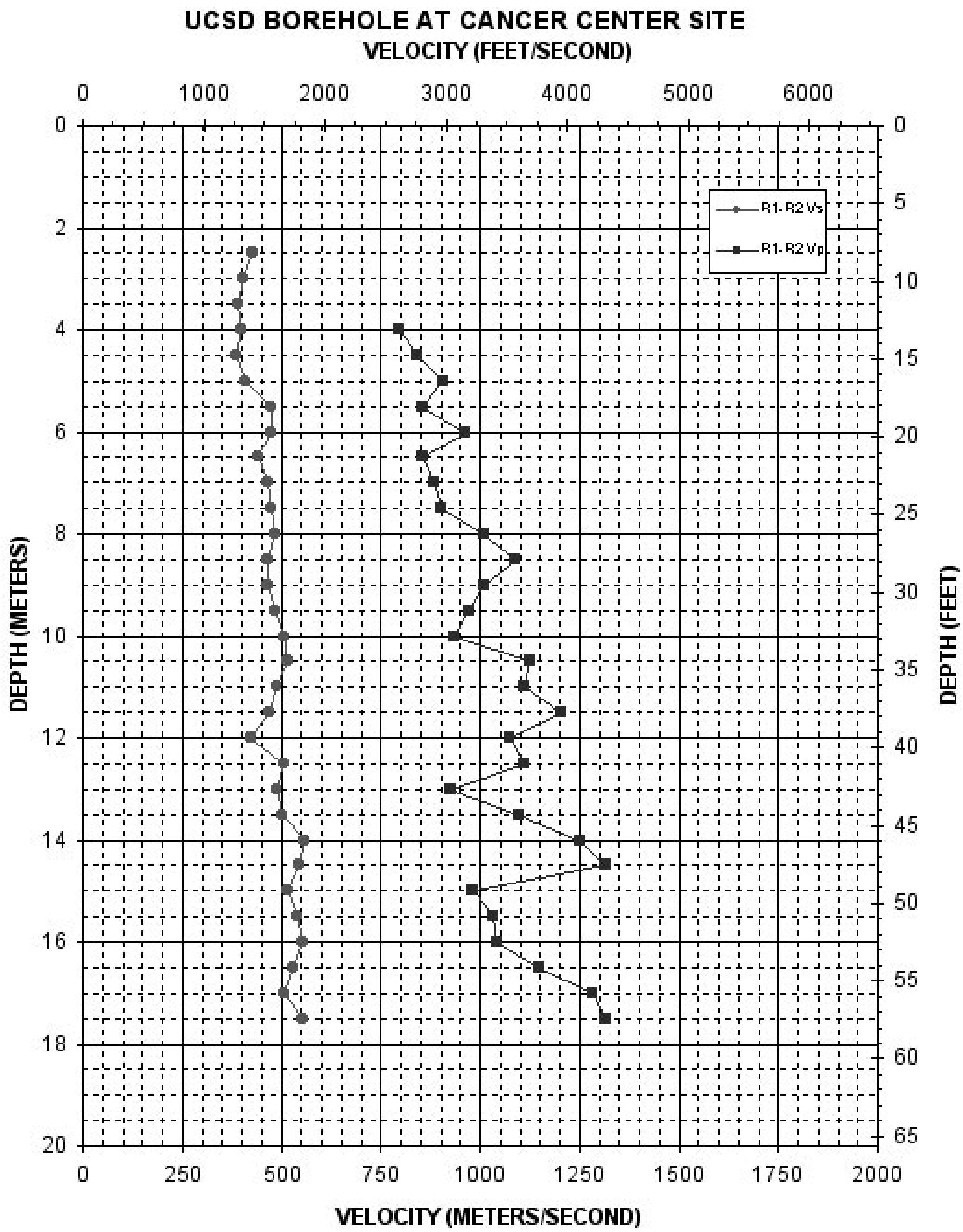


Figure 3.1 P- and S-wave velocity profiles at the Cancer Center

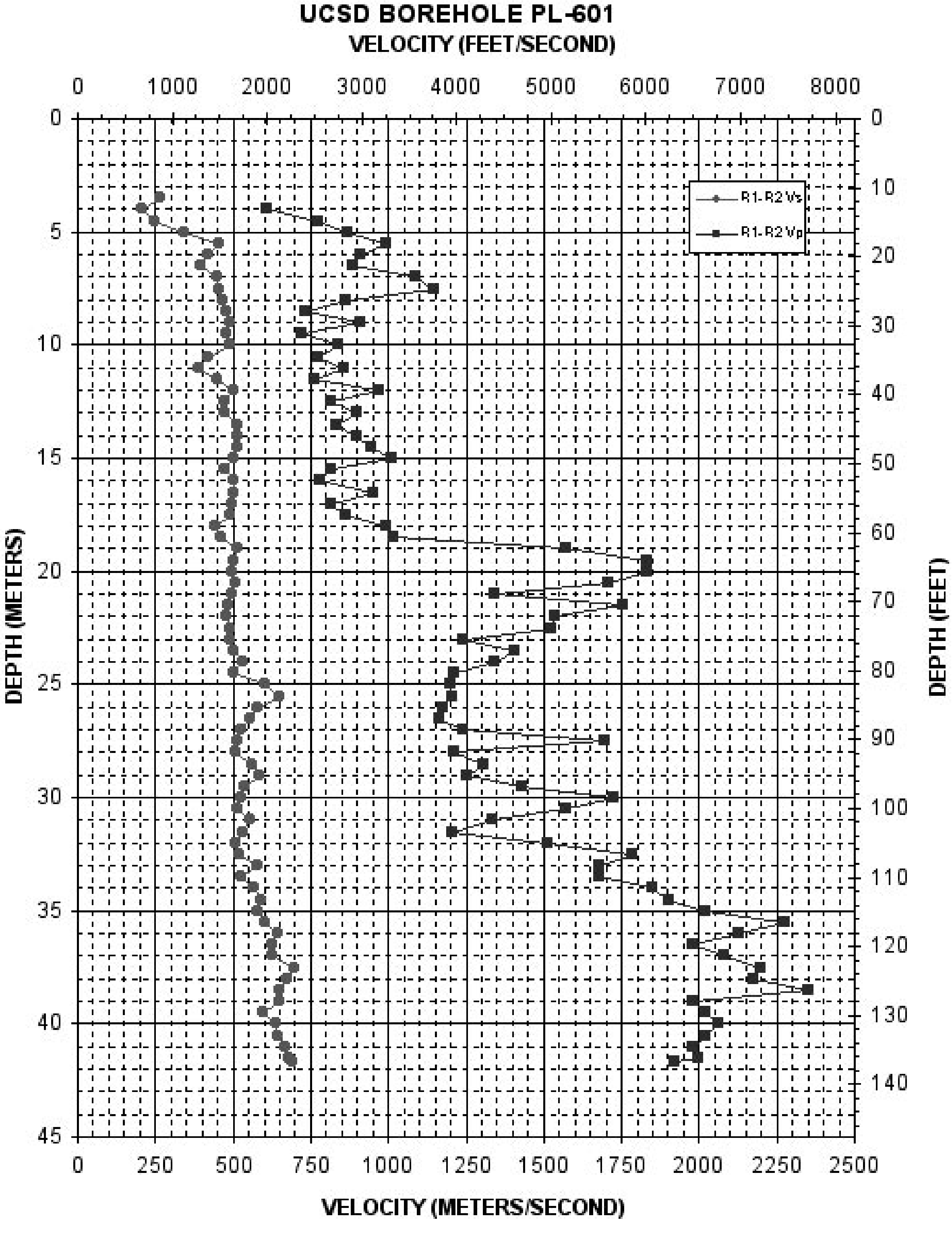


Figure 3.2 P- and S-wave velocity profiles at Parking Lot 601

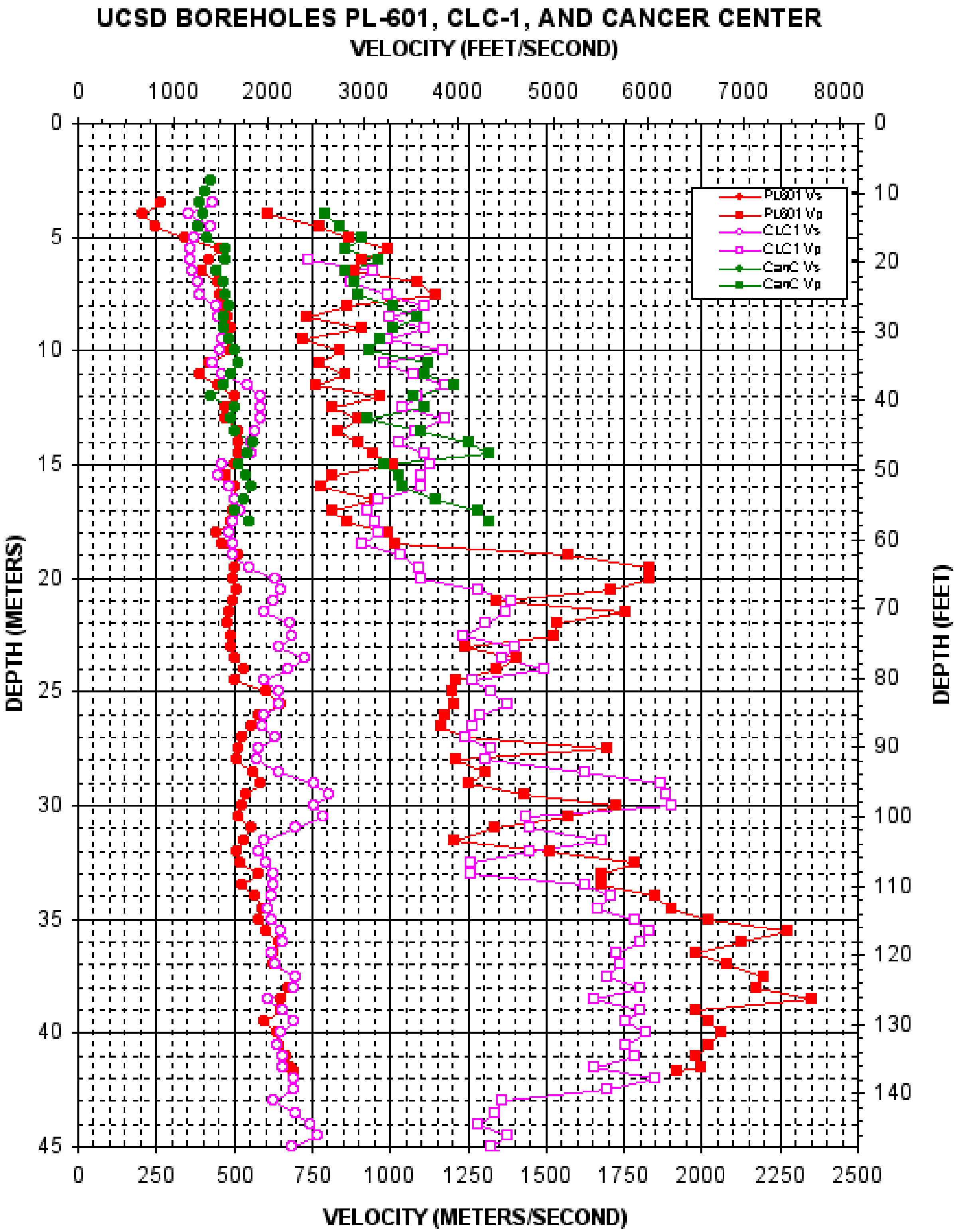


Figure 3.3 Comparison of P- and S-wave velocity plots for the three UCSD sites (CLC 1 is the Thornton hospital site)

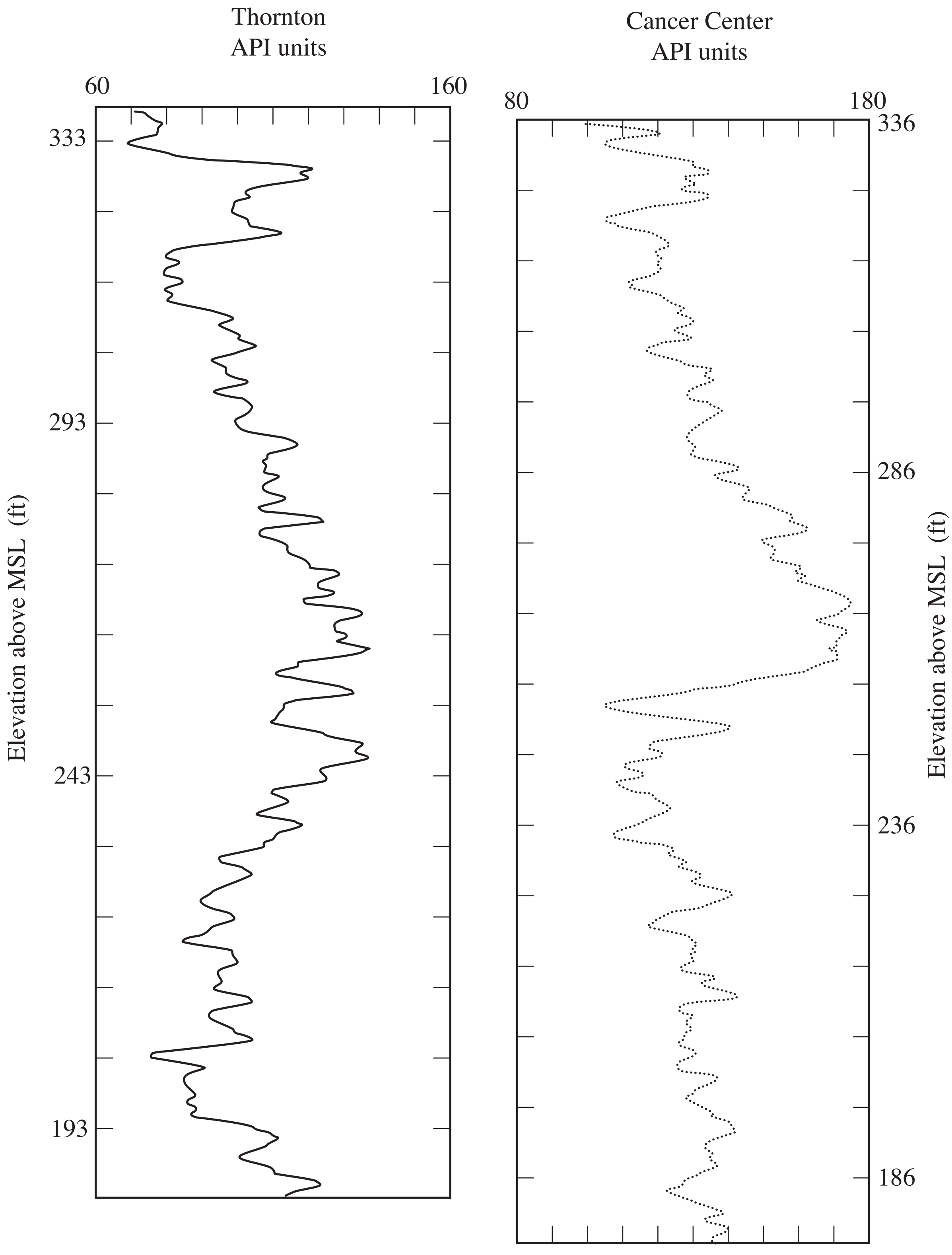


Figure 3.4 Comparison of gamma logs for the Thornton and Cancer Center sites. (English units have been retained, as they were in the original logs)

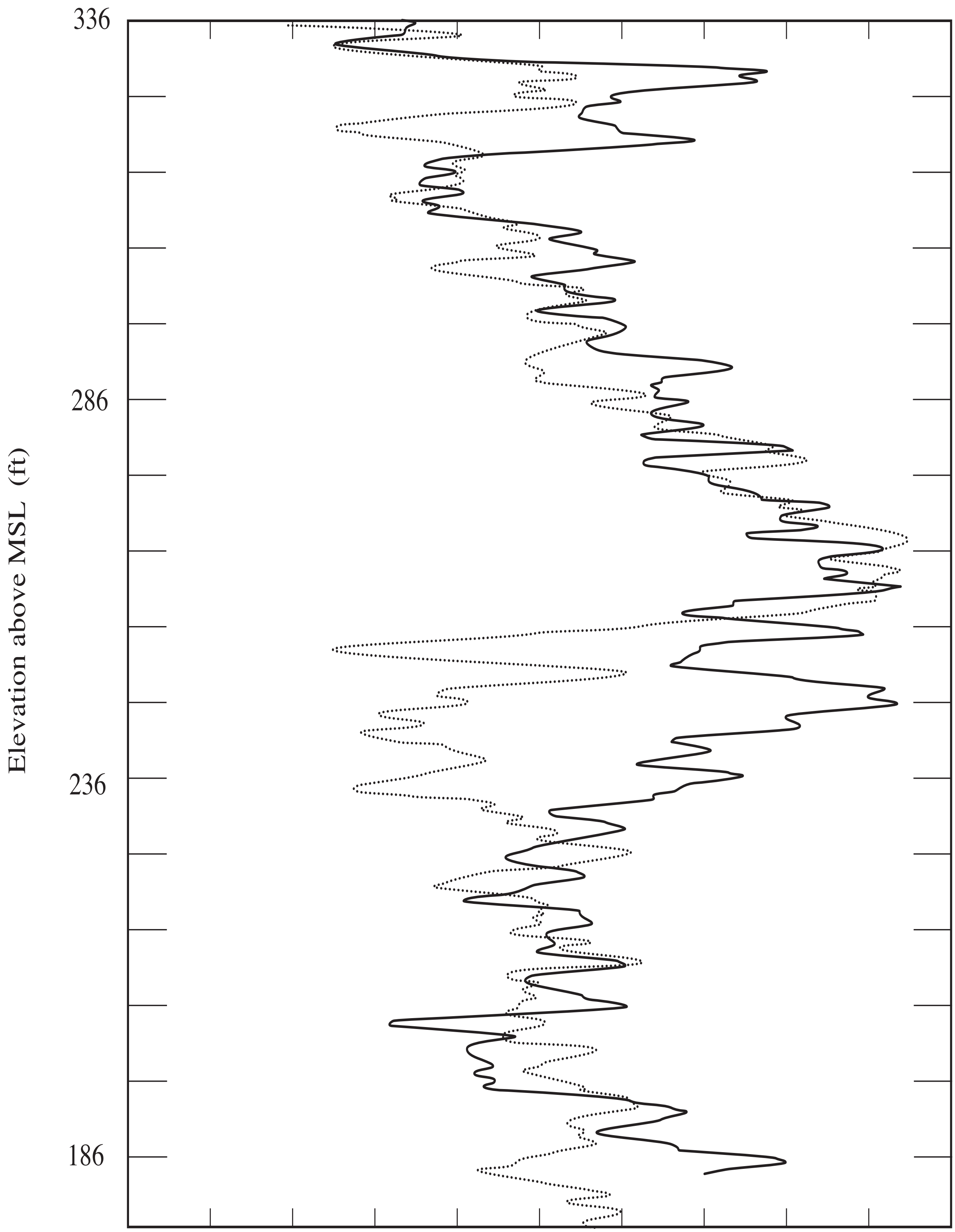


Figure 3.5 Superposition of the gamma logs at Thornton and at the Cancer Center. (English units have been retained, as they were in the original logs)

4.0 SOIL DYNAMICS STUDIES

4.1 Laboratory Tests on UCSD Soils

Soil samples were recovered by Shelby tubes, at the location of the seismic station. In order to complement the in-situ characterization tests and to obtain properties required for soil dynamics calculations, laboratory tests were performed on the samples. Soil classification and cyclic simple shear tests were conducted at the University of California at Los Angeles (UCLA), and monotonic triaxial tests were done at the University of California at Berkeley (UCB). The detailed test results are presented in Vucetic et al (1999), and in Riemer and Abu-Safaqah (1999), respectively. Only a summary is given here.

4.1.1 Basic Soil Properties and Soil Classification

The results of basic and soil classification tests done at UCLA are summarized in Table 4.1 and Figure 4.1.

Table 4.1 Basic properties and Classification of soils from the UCSD Thornton site

Sample label	Depth (m)	LL*	PI*	Soil classification	Dry unit weight (kN/m^3)	Water content (%)	Void ratio	Saturation of test samples (%)
SD-6	1.8	26.9	5.4	CL-ML sandy silty clay	14.0	8.9	0.90	27
SD-20	6.1	51.8	35.2	CH - fat clay	16.4	21.8	0.62	95
SD-22	6.7	51.0	33.7	CH – fat clay	16.3	23.0	0.62	99
SD-47	14.2	57.4	33.3	CH – fat clay	16.9	20.9	0.60	97
SD-67	20.4	46.2	25.8	CL – lean clay	16.4	18.1	0.65	77
SD-122	37.2	48.8	25.8	CL – lean clay	17.4	18.6	0.55	93
SD-299	91.1	49.1	24.2	CL – lean clay	18.6	15.2	0.45	93

* LL : Liquid Limit, PI : Plasticity Index

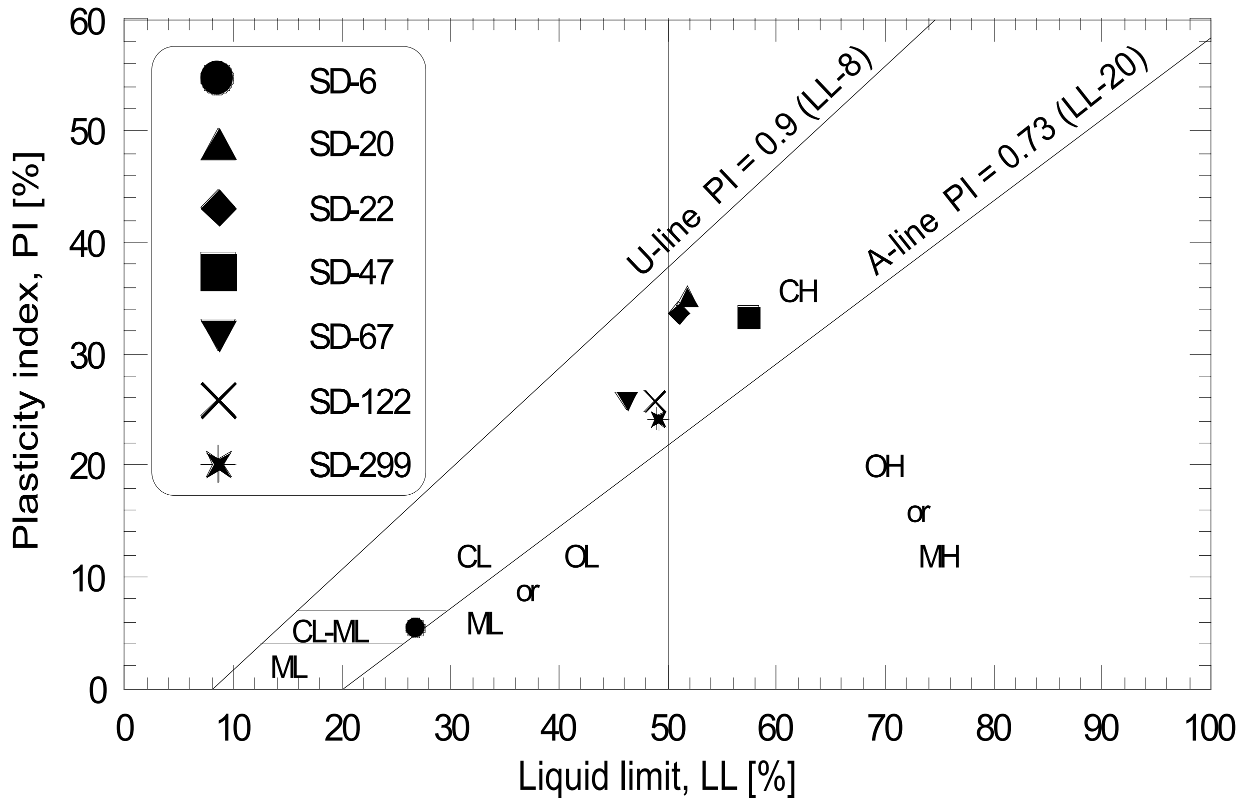


Figure 4.1 Classification of UCSD Thornton soils, based on Liquid Limit and Plastic Limit

4.1.2 Cyclic Simple Shear Tests

These tests were conducted in the Civil Engineering Department at UCLA. The device used was designed by Doroudian and Vucetic (1995). As shown in Figure 4.2, its most unique feature is that two parallel specimens of the same soil are tested simultaneously. Such a special configuration enables almost complete elimination of problems associated with false deformation, system compliance, and friction. As a result, very small strains can be applied and measured in a controlled manner, as well as the resulting stresses. The cyclic response of the soil samples is recorded in terms of the variation of shear stress vs. shear strain over numerous cycles of loading with increasing strain amplitude. From these records one can describe the progressive decay of soil shear modulus (G) and the increase in the equivalent viscous damping ratio (λ). The definitions of these quantities are illustrated in Figures 4.3 and 4.4.

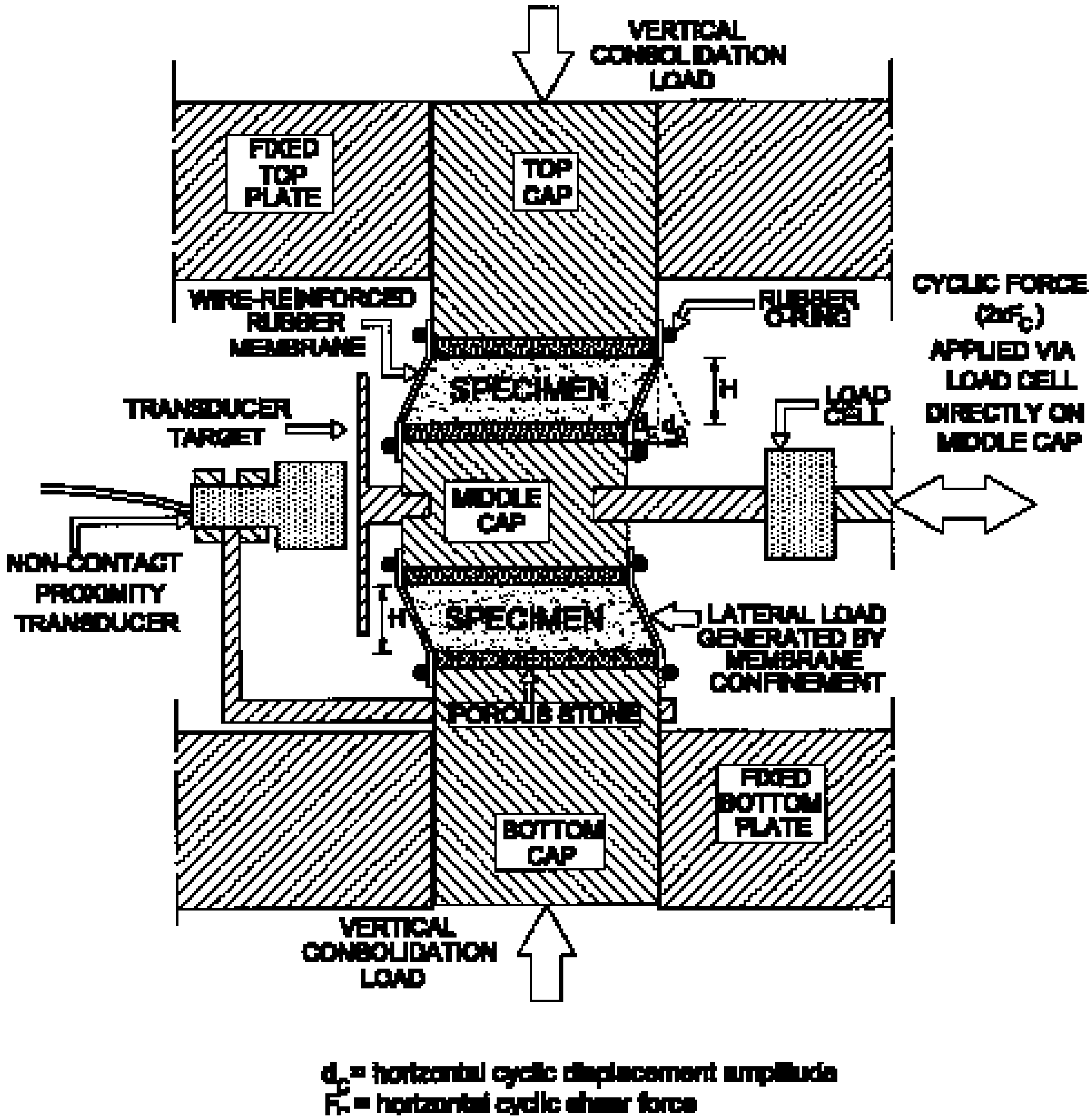


Figure 4.2 Schematic of the UCLA Double Simple Shear system

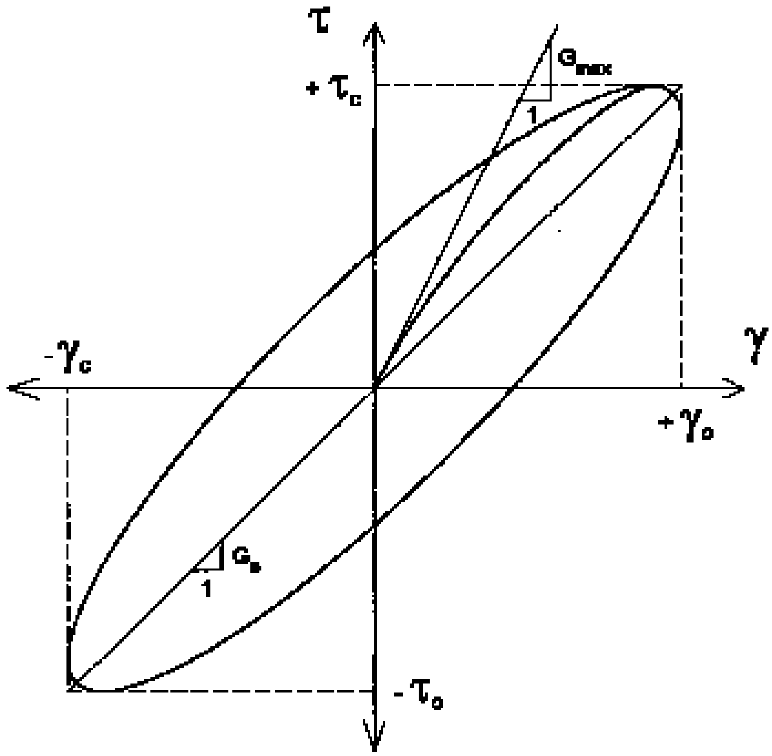


Figure 4.3 Idealized stress-strain loop during cyclic shearing, with parameter definition

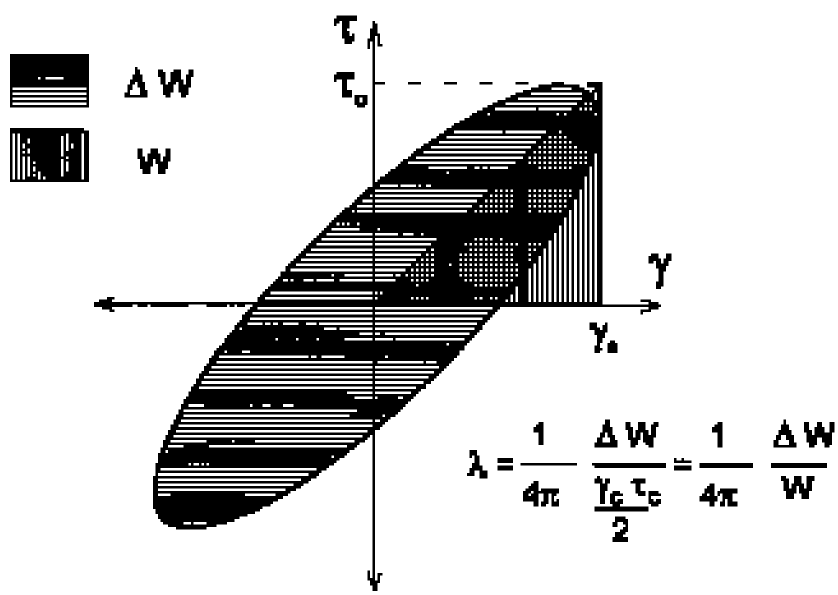


Figure 4.4 Definition of the equivalent viscous damping ratio used in this study

Seven samples, recovered from depths between 1.8 m and 91.1 m, were tested in a cyclic strain-controlled mode. The cyclic frequency was lower than 0.25 Hz. The dynamic properties of cohesionless soils are practically independent of loading frequency (Hardin, 1965), and tests on cohesive soils have shown the effect of frequency to be small so that it can be negligible (Kramer et al, 1992).

The test results are summarized in Figure 4.5. The variation of shear modulus with shear strain was measured over a broad range of strains. Values of the equivalent viscous damping ratio were also obtained for shear strains down to at least10^{-5}, and up to 10^{-3} in most cases.

The maximum shear modulus measured in the laboratory can be compared to that obtained from in-situ shear-wave velocity logs (Gmax). At UCSD's Thornton site, for the seven locations where this comparison can be performed, between depths of 1.8 m and 91.1 m, the ratio of laboratory to field values is between 0.07 and 0.30 (Table 4.2). In this study, the fact that the level of in-situ stresses could not be replicated in the UCLA shear system contributed to the lowering of laboratory Gmax. Additional lowering can be attributed to the transfer of soil samples from the ground to the laboratory testing system.

For nonlinear soil dynamics computations, the laboratory moduli at the lowest strain (10^{-6}) are set to the value of Gmax, and the rest of the shear-strain shear-modulus curve is normalized to this Gmax value. This is based on the premise that the field values are representative of the properties of the undisturbed material. This procedure, commonly used in geotechnical engineering, has recently been compared by others investigators to several possible laboratory-to-field adjustments and was recommended as the best (Pitilakis and Anastasiadis, 1998).

Table 4.2: Ratio of laboratory Gmax to field Gmax for UCSD soils

Depth (m)	Laboratory Gmax (MPa)	Field Gmax (MPa)	Ratio, Laboratory/Field
1.8	12	171	0.07
6.1	78	259	0.30
6.7	69	274	0.25
14.2	138	631	0.22
20.4	118	803	0.15
37.2	212	893	0.24
91.1	430	1649	0.26

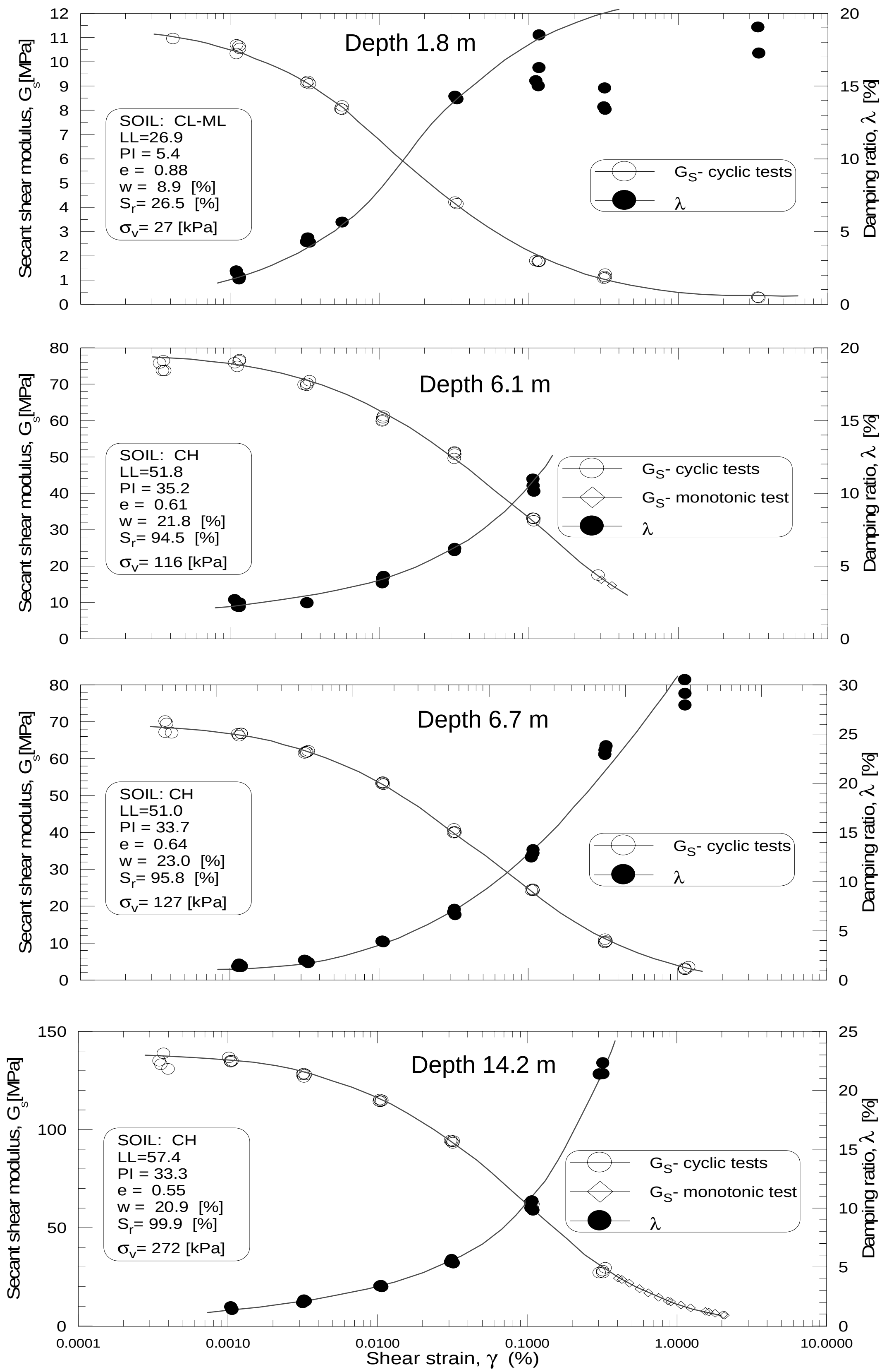


Figure 4.5 Summary of simple shear test results on soils from the Thornton site

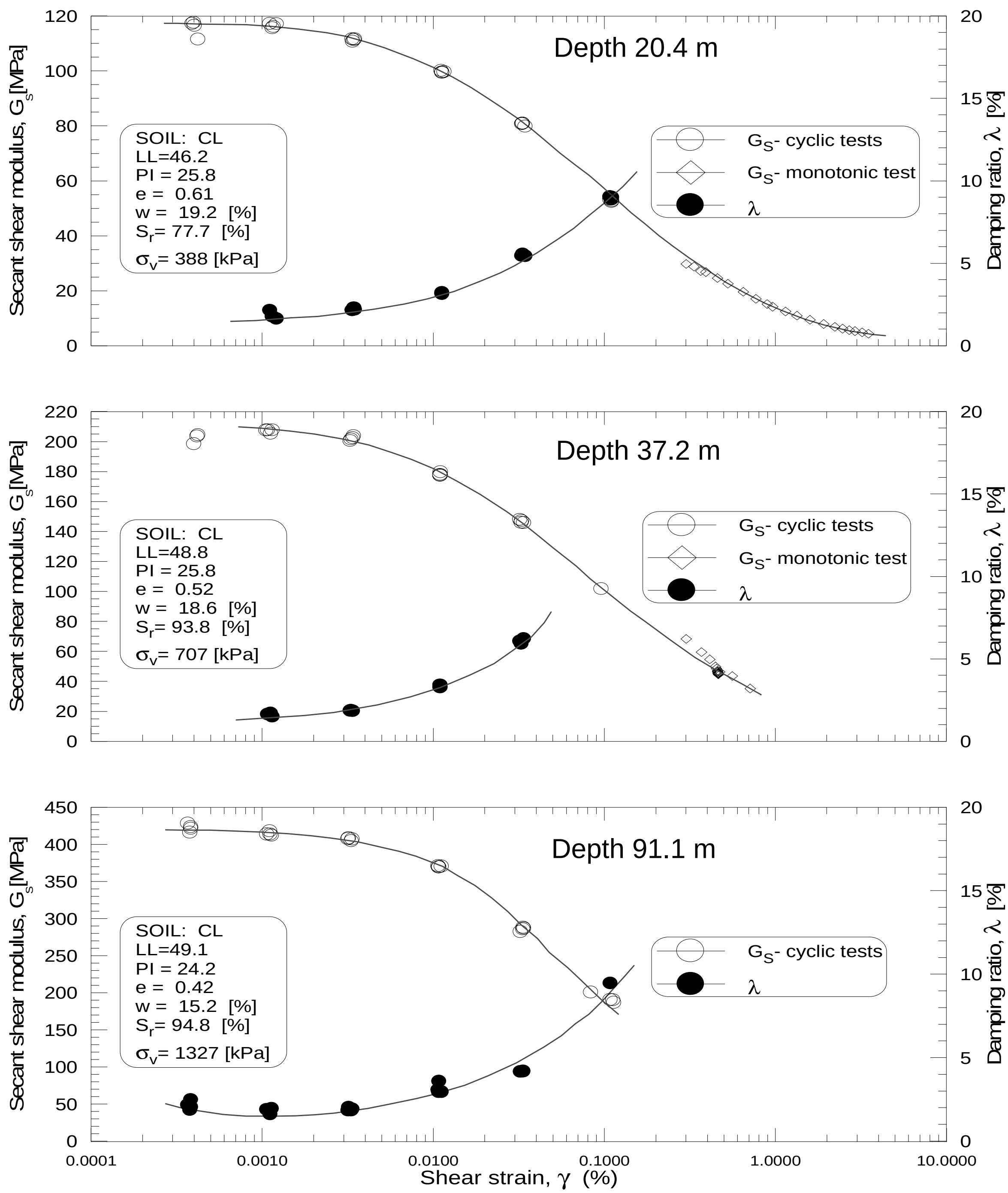


Figure 4.5 (cont.)

4.1.3 Monotonic Triaxial Tests

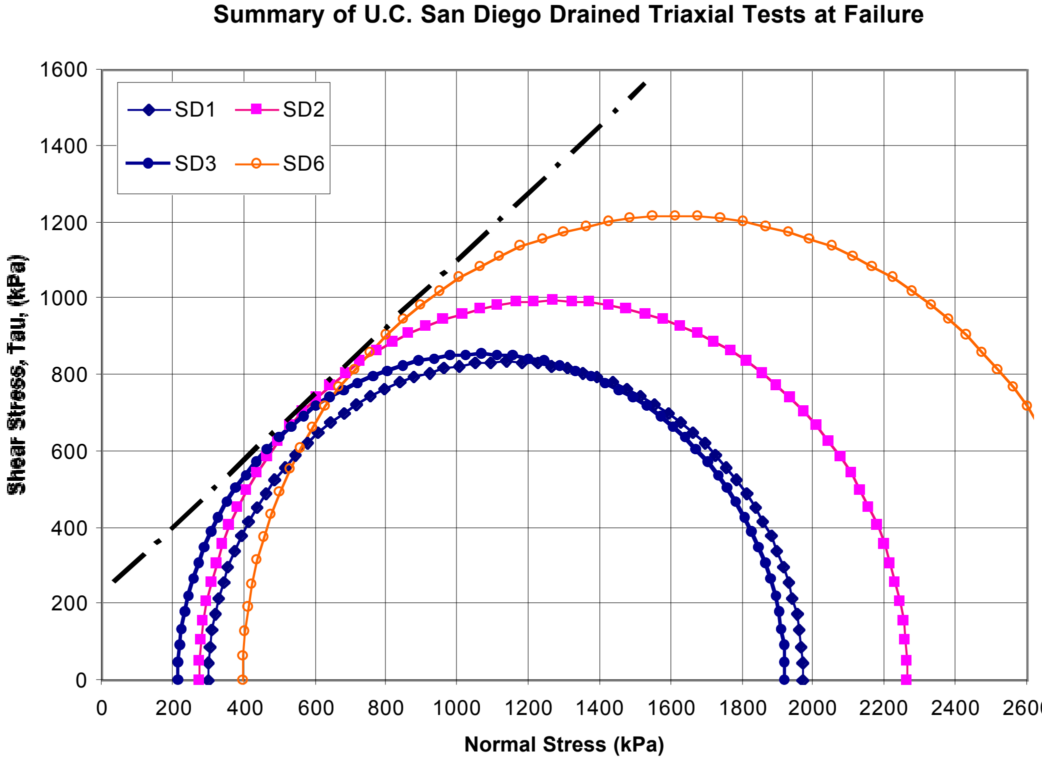
These tests were conducted in the Geotechnical Laboratory of the Department of Civil Engineering at U.C. Berkeley. The U.C. Berkeley triaxial testing system is shown in Figure 4.6. All the tests were performed in drained conditions, using a strain-control mode.

Figure 4.6 The U.C. Berkeley triaxial testing system

Because of the limited number of samples, the UCLA and Berkeley tests were performed on soils from different horizons. The depths of the Berkeley samples are as follows:

Sample	SD 1	SD 2	SD 3	SD 4	SD 5	SD 6	SD 7
Depth (m)	19.7	13.6	5.1	3.6	30.0	45.4	36.4

The triaxial failure envelopes of Figure 4.7 show the samples in two distinct groups. The weaker envelope (SD 4, 5, 7) indicates a cohesion of approximately 150 kPa and a friction angle of 27 degrees. The stronger envelope (SD 1, 2, 3, 6) shows a cohesion of 200 kPa and a friction angle of about 40 degrees. Since the soils are moderately plastic and believed to be quite overconsolidated in the field, the large values of cohesion are not surprising. It is likely that the higher friction angle more closely reflects the field properties since two of the weaker specimens were from larger depths and may have experienced substantial disturbance form the release of large lateral stresses.



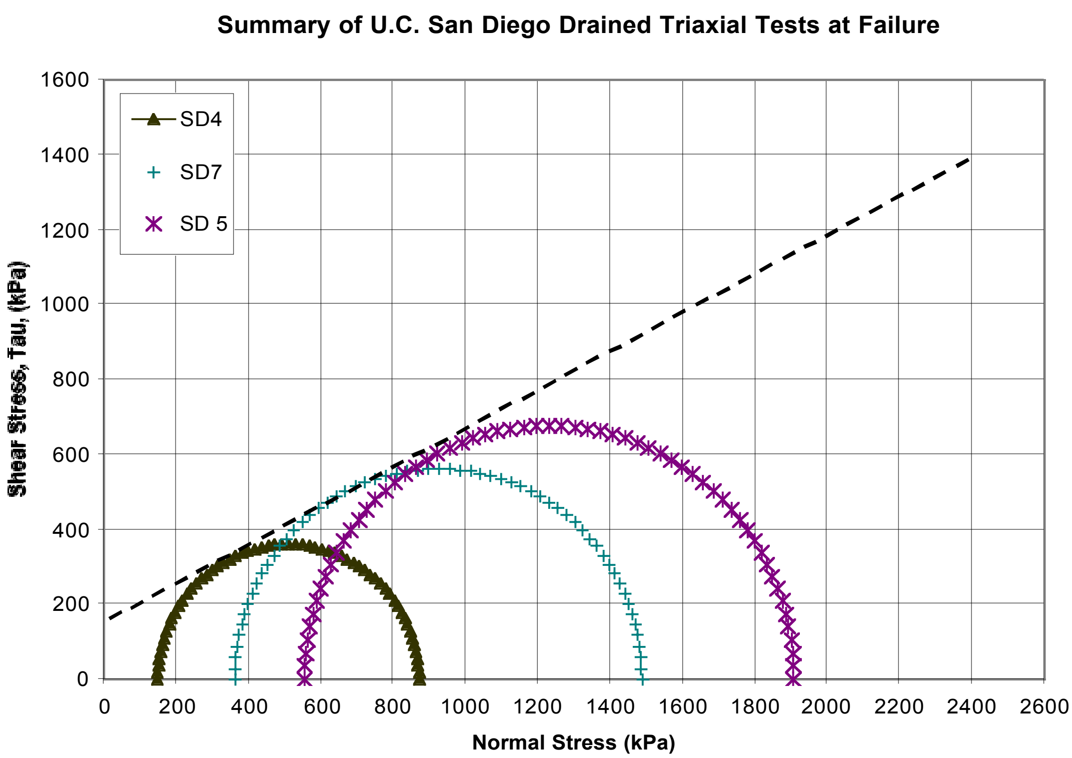


Figure 4.7 Failure envelopes for UCSD soil samples

4.2 Soil Dynamics Computational Models

4.2.1 The CYCLIC Soil Dynamics Computer Model

In order to study the dynamic response of saturated soil systems as an initial-boundary-value problem, a numerical code CYCLIC was developed to couple the solid and fluid phases. CYCLIC (Parra 1996, Yang 2000, Elgamal et al. 2000) is a two-dimensional (2D plane-strain and axisymmetric) Finite Element program that implements the two-phase, fully coupled numerical formulation of Chan (1988) and Zienkiewicz et al. (1990). The CYCLIC code incorporates a material constitutive model developed for liquefaction analysis (Parra 1996, Yang et al. 2000). In the following sections, the adopted finite element and constitutive model formulations are summarized.

4.2.1.1 Finite element formulation

Soil is modeled as a two-phase material based on the Biot (1962) theory for porous media. A simplified numerical formulation of this theory, known as u-p formulation, in which the displacement of the soil skeleton u, and pore pressure p, are the primary unknowns (Zienkiewicz et al. 1990), was implemented in CYCLIC (Ragheb 1994, Parra 1996, Yang 2000). The computational scheme follows the methodology of Chan (1988), that is based on the following assumptions: i)the deformations are small and the rotations are negligible, ii) the densities of the solid and fluid are constant in both time and space, iii) porosity is locally homogeneous and constant with time, iv) soil grains are incompressible, and v) accelerations for the solid and fluid phases are equal. The u-p formulation is defined by (Chan 1988): i) an equation of motion for the solid-fluid mixture, and ii) an equation of mass conservation for the mixture, incorporating equation of motion for the fluid phase and Darcy's law:

$$\nabla \bullet (\sigma - p\delta) - \rho(\ddot{\mathbf{u}} - \mathbf{g}) = \mathbf{0} \tag{1a}$$

$$\frac{\dot{p}}{Q} + \nabla \bullet \dot{\mathbf{u}} - \nabla \bullet \left[\frac{\mathbf{k}}{\rho_f g} \left(\nabla p + \rho_f \ddot{\mathbf{u}} - \rho_f \mathbf{g} \right) \right] = 0 \tag{1b}$$

where σ is the effective stress tensor, p is pore-fluid pressure, δ is the second-order identity tensor, ρ is mass density of the mixture, $\mathbf{u}$ is the displacement vector of the solid phase, $\mathbf{g}$ is gravity acceleration vector, g is the absolute value of gravity acceleration, Q is the undrained mixture bulk modulus, $\mathbf{k}$ is Darcy's permeability coefficient tensor, ρ_f is fluid mass density, ∇ is the gradient operator, $\nabla\bullet$ is the divergence operator, and a superposed dot denotes material derivative. After finite element spatial discretization and Galerkin approximation, the governing equations can be expressed in the following matrix form (Chan 1988):

$$\mathbf{M\ddot{U}} + \int_{\Omega} \mathbf{B}^{\mathbf{T}} \sigma \mathbf{d}\Omega - \mathbf{Qp} - \mathbf{f}^{s} = \mathbf{0} \tag{2a}$$

$$\mathbf{Q}^{\mathbf{T}} \mathbf{\dot{U}} + \mathbf{Hp} + \mathbf{S\dot{p}} - \mathbf{f}^{\mathbf{p}} = \mathbf{0} \tag{2b}$$

where **M** is the mass matrix, **U** the displacement vector, **B** the strain-displacement matrix, _ the effective stress vector (determined by the soil constitutive model discussed below), **Q** the discrete gradient operator coupling the solid and fluid phases, p the pore pressure vector, **H** the permeability matrix, and **S** the compressibility matrix. The vectors $\mathbf{f}^s$ and $\mathbf{f}^p$ include the effects of body forces and the prescribed boundary conditions for the solid and fluid phases respectively. It is clearly seen from Eq. 2b that high material permeability corresponds to a large-valued **H** (permeability) matrix, which numerically behaves as a penalty term that forces pore pressure **p** changes to be negligibly small. In CYCLIC, Rayleigh viscous damping can be added in the form of $\mathbf{C} = \alpha\mathbf{M} + \beta\mathbf{K}$, where **K** is the solid-phase initial stiffness matrix.

4.2.1.2 Constitutive model

In the developed liquefaction model, emphasis is placed on controlling the magnitude of cycle-by-cycle permanent shear strain accumulation in clean medium-dense sands (Parra 1996, Yang *et al*. 2000). Specifically, the experimentally observed accumulation of permanent shear strain was modeled by using strain-space parameters (Yang 2000), within a multi-yield surface stress-space model (Prevost 1985). Furthermore, appropriate loading-unloading flow rules were devised to reproduce the observed strong dilation tendency, and resulting increase in cyclic shear stiffness and strength. The main components of this model are summarized below.

Yield function: Following the classical plasticity convention (Hill 1950), it is assumed that material elasticity is linear and isotropic, and that nonlinearity and anisotropy result from plasticity. The selected yield function (Prevost 1985, Lacy 1986) forms a conical surface in stress space with its apex along the hydrostatic axis. In the context of multi-yield-surface plasticity (Iwan 1967, Mroz 1967, Prevost 1985), a number of similar yield surfaces with a common apex and different sizes form the hardening zone. The outmost surface is designated as the failure surface.

Hardening rule: A purely deviatoric kinematic hardening rule (Prevost 1985) is employed to account for the Bauschinger effect exhibited by soil under cyclic loading. All yield surfaces but the outermost may translate in stress space (Parra 1996, Yang 2000).

Flow rule: During shear loading, the soil contractive/dilative behavior is handled by a non-associative flow rule (Parra 1996) so as to achieve appropriate interaction between shear and volumetric response. In particular, nonassociativity is restricted to the volumetric component P″ of the plastic flow tensor (outer normal to the plastic potential surface in stress-space). Therefore, depending on the relative location of the stress state (Figure 4.8) with respect to the phase transformation (PT) surface (Ishihara 1985, Vaid and Thomas 1995, Vaid and Sivathayalan 1999, Iai 1991, 1998, Dobry and Abdoun 1998, Kramer 1996, Kramer and Arduino 1999), different expressions for P″ were specified for (Parra 1996): i) the contractive phase, when the stress-state lies inside the PT surface (Fig. 4.8, phase 0-1), ii) the dilative phase during loading, if the stress-state lies outside the PT surface (Fig. 4.8, phase 2-3), and iii) the contractive phase during unloading, with the stress-state outside the PT surface (Fig. 4.8 , phase 3-4).

At low effective confining pressure, when the stress state reaches the PT surface while loading, permanent shear strain may accumulate rapidly with essentially no change in shear stress (Figure 4.8, phase 1-2). This is achieved by activating a perfectly plastic zone (PPZ, Figure 4.8, phase 1-2) before the initiation of dilation outside the PT surface (Figure 4.8, phase 2-3). The PPZ is defined in deviatoric strain space as a circular, initially isotropic surface (Yang et al. 2000). Depending on the current strain state and plastic loading history, the PPZ may enlarge and/or translate in deviatoric strain space to model the accumulation of permanent shear deformations (Yang et al. 2000). More information on CYCLIC is available at http://casagrande.ucsd.edu.

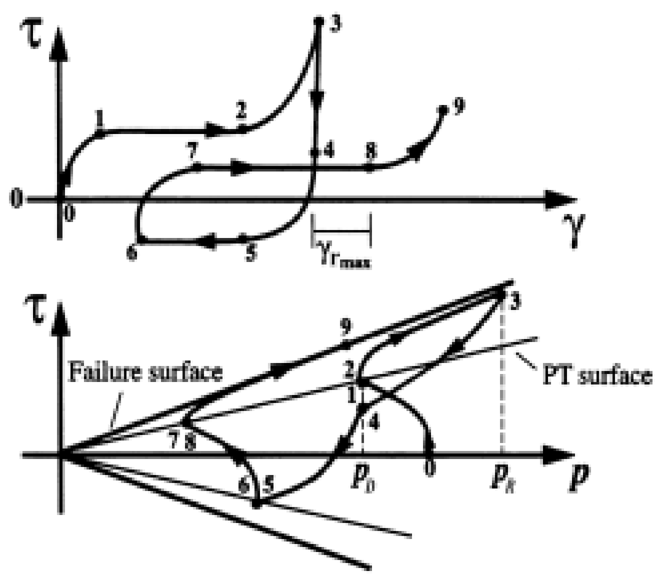


Figure 4.8 Schematic of constitutive model response showing the octahedral stress τ, the effective confinement pressure p, and the octahedral strain γ relationship (after Parra, 1996)

4.2.2 Comparison of CYCLIC with Other Nonlinear Soil Models

The field of nonlinear dynamic analysis is much more complex than that of linear analysis. It behooves calculators to make every effort to verify their nonlinear calculations. Since analytical, exact solutions are very scarce for such purpose, an accepted practice is to compare the results obtained with different nonlinear models. In order to assess the calculations performed with the CYCLIC model, the CEP also took advantage of the availability of other soil dynamics models in the U.C. community. Two such U.C. codes were exercised. Both perform three-component one–dimensional wave propagation through nonlinear soils. The first is SUMDES (Sites Under Multi-Directional Earthquake Shaking). from U.C. Davis (Li et al, 1992). Its formulation is based on “bounded surface” plasticity and it can do effective stress analysis. The second is NOAH, from U.C. Santa Barbara (Bonilla et al., 1998). It is an effective stress formulation, as well. The excellent agreement among the calculations with the three different models is described in detail in the Phase 2 report for U.C. Santa Barbara (Archuleta et al., 2000a) and will not be repeated in full here. An example of the comparison is given in Figure 4.9.

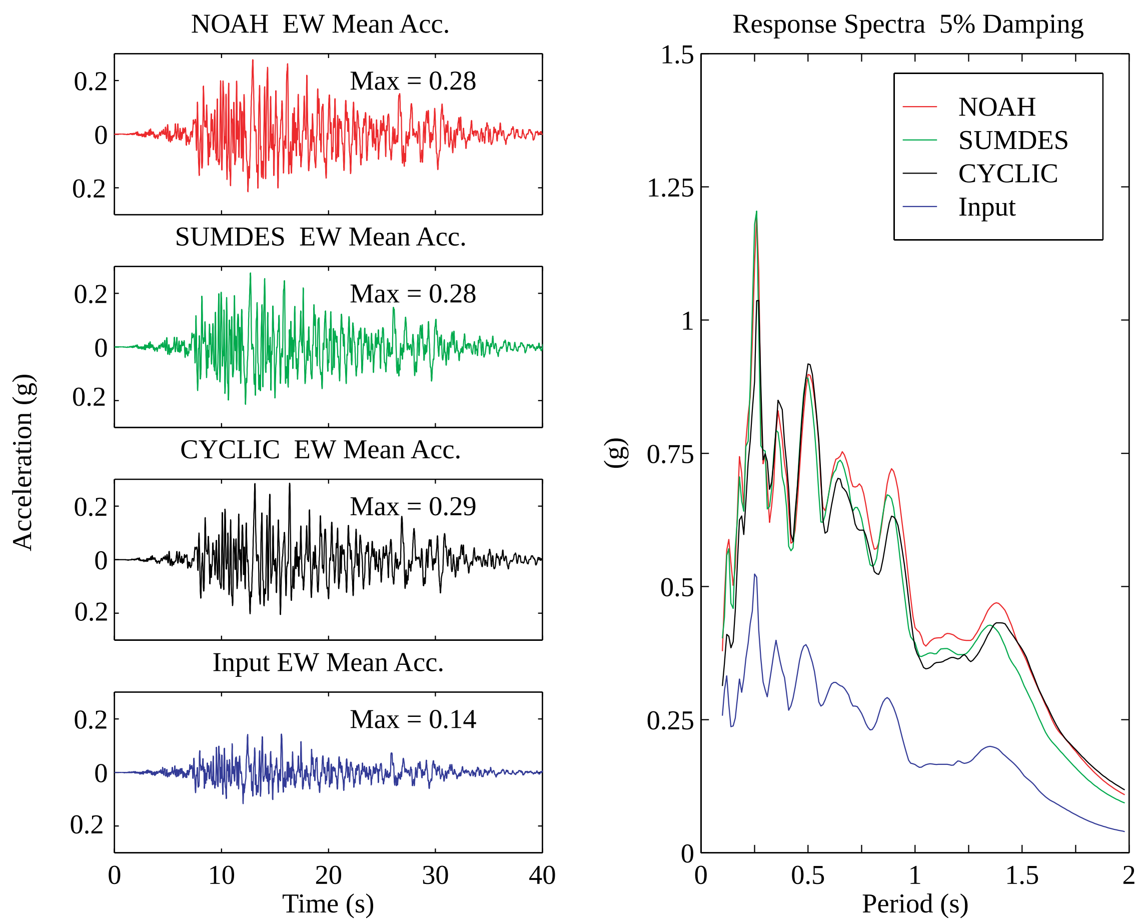


Figure 4.9 Comparison of NOAH, SUMDES, and CYCLIC results for a strong motion study of U.C. Santa Barbara (after Archuleta et al., 2000a).

4.2.3 UCSD Computational Soil Profiles at Thornton Hospital/Cancer Center and PL 601

The soil profiles for the Thornton and PL 601 sites were derived primarily from the suspension velocity logs, with additional input from the gamma and resistivity logs. The layers and the soil properties defined for the soil dynamics calculations are summarized in Tables 4.3 and 4.4. For all layers, the coefficient of earth pressure at rest is K 0 =0.5, and the angle of internal friction is $\phi = 36^{o}$.

Table 4.3 Computational soil profile at the Thornton hospital/Cancer Center site

Depth (m)	Vs (m/s)	Vp (m/s)	Unit weight (kN/m^3)	Damping (%)
4	416	870-	18.4	2.0
8	410	870	18.4	2.0
11	480	1010	18.4	2.0
14.5	600	1086	18.4	2.0
16.5	520	1160	20.5	2.0
20	520	1030	20.5	2.0
28	630	1290	19.4	2.0
31	805	1750	19.4	2.0
33	610	1450	19.4	2.0
42.5	655	1740	19.4	2.0
48.5	685	1350	20.6	2.0
50	840	1740	20.6	2.0
51.5	750	2050	20.6	2.0
54.5	1050	2700	20.6	2.0
59	695	1928	20.6	2.0
61	1020	2380	20.6	2.0
64.5	716	1911	20.6	2.0
65.5	900	2040	20.6	2.0
68.5	720	1815	20.6	2.0
70.5	820	2100	20.6	2.0
73	820	2020	20.6	2.0
78	735	1870	20.6	2.0
81	840	2000	20.6	2.0
83	895	1930	20.6	2.0
84	966	2202	20.6	2.0
91	850	2020	20.6	2.0

Table 4.4 Computational soil profile at the Medical Building/Parking Lot 601 site

Depth (m)	Vs (m/s)	Vp (m/s)	Unit weight (kN/m3)	Damping (%)
4.5	250	605	18.4	2.0
7	415	885	18.4	2.0
8	470	980	18.4	2.0
13	485	840	18.4	2.0
17.5	520	900	20.5	2.0
19	465	1020	20.5	2.0
23	510	1570	19.4	2.0
24.5	565	1350	19.4	2.0
26.5	645	1240	19.4	2.0
28	550	1630	19.4	2.0
29.5	585	1300	19.4	2.0
32	550	1515	19.4	2.0
34	550	1740	19.4	2.0
35	590	1905	19.4	2.0
39	675	2150	19.4	2.0
40	635	2020	19.4	2.0
42	678	1980	19.4	2.0

4.3 The CEP Surface Strong Motion Estimates

4.3.1 Calculations of Surface Strong Motions for the Thornton/Cancer Center site

The set of 300 time-histories of incident downhole motions were propagated from a depth of 91 m to the surface through the nonlinear soils with the CYCLIC model. The results are shown on Figures 4.10 in terms of spectral accelerations vs. period. The three spectral lines are respectively the mean and the plus and minus one standard deviation of the scenario population. Acceleration time-histories representative of the mean and + 1 sigma scenarios are shown in Figure 4.11.

4.3.2 Calculations of Surface Strong Motions for the Medical Building/PL 601 site

The set of 300 time-histories of incident downhole motions were propagated from a depth of 42 m to the surface through the nonlinear soils with the CYCLIC model. The results are shown on Figures 4.12 in terms of spectral accelerations vs. period. The three spectral lines are respectively the mean and the plus and minus one standard deviation of the scenario population. Acceleration time-histories representative of the mean and + 1 sigma scenarios are shown in Figure 4.13.

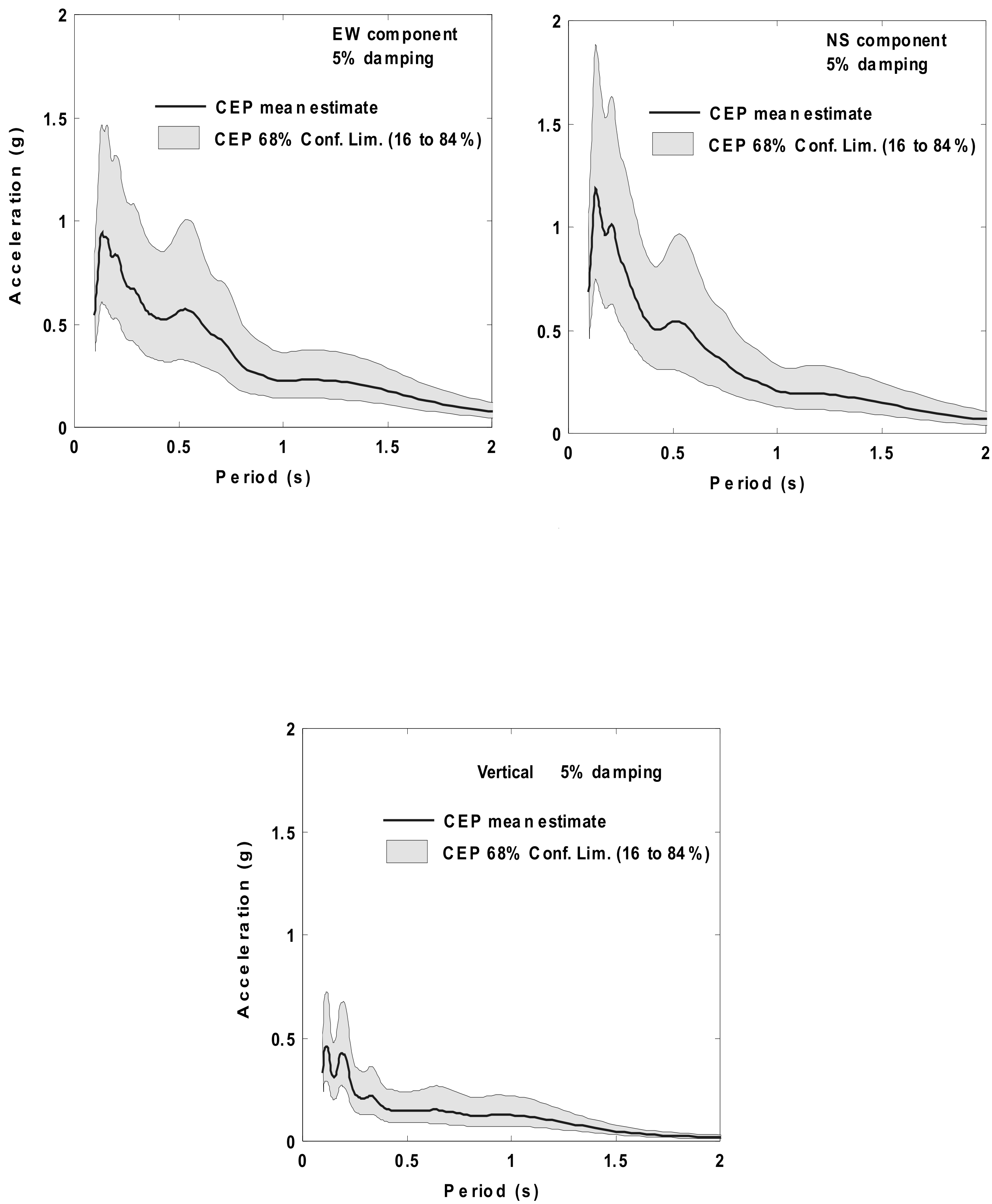


Figure 4.10 CEP surface acceleration response spectra at the Thornton /Cancer Center site

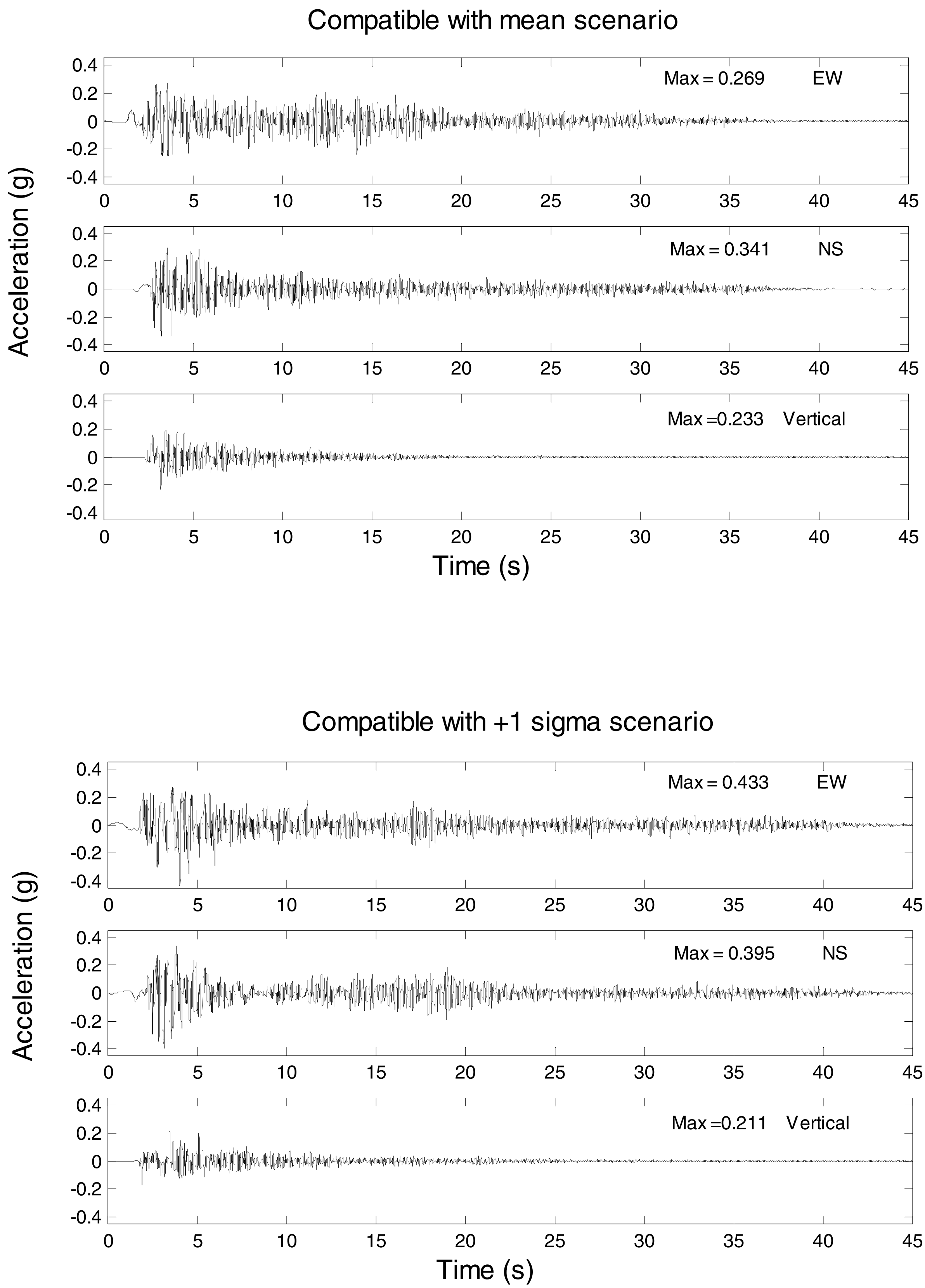


Figure 4.11 Representative acceleration time-histories for the Thornton/Cancer center site

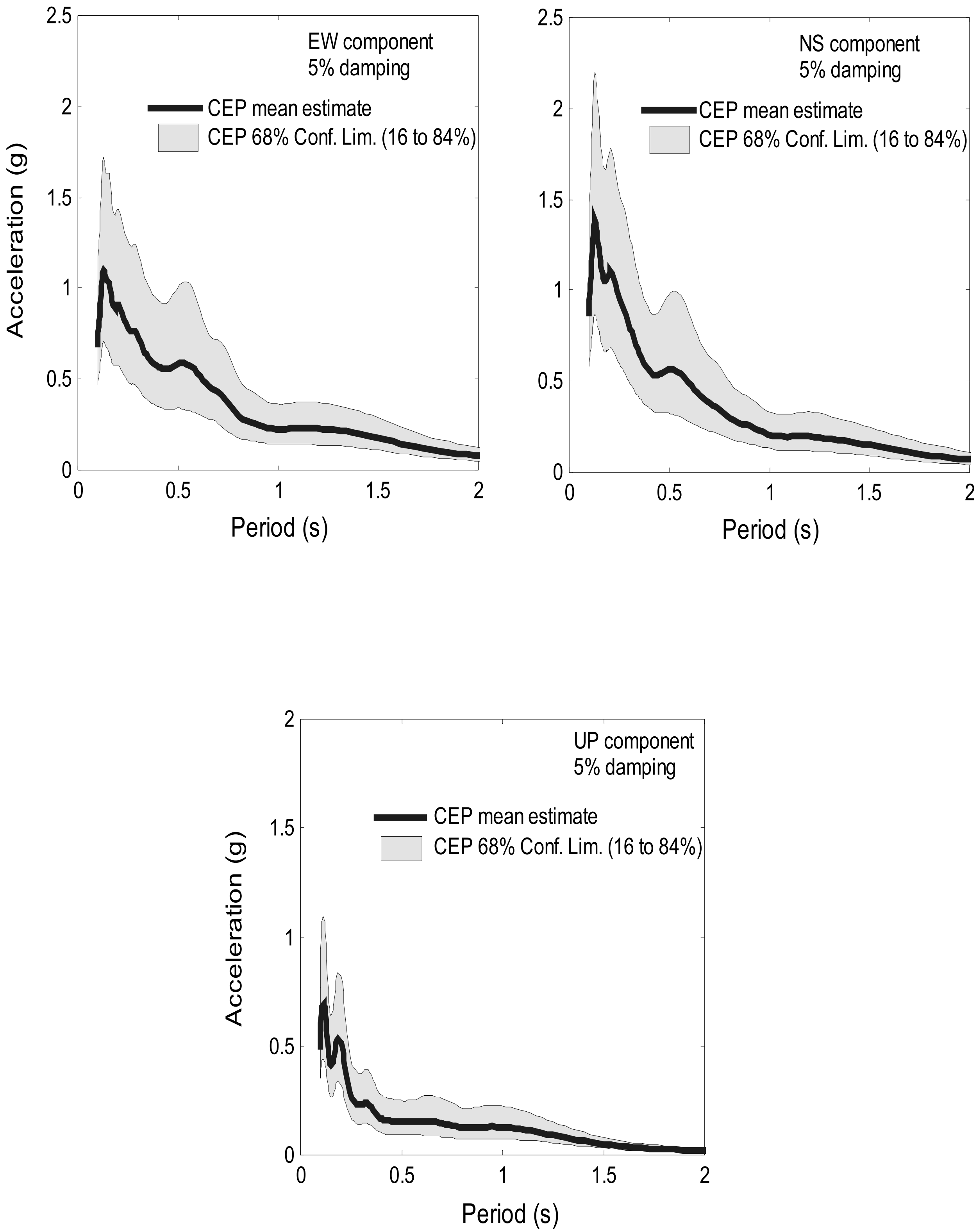


Figure 4.12 CEP surface acceleration response spectra at the Medical building/PL 601 site

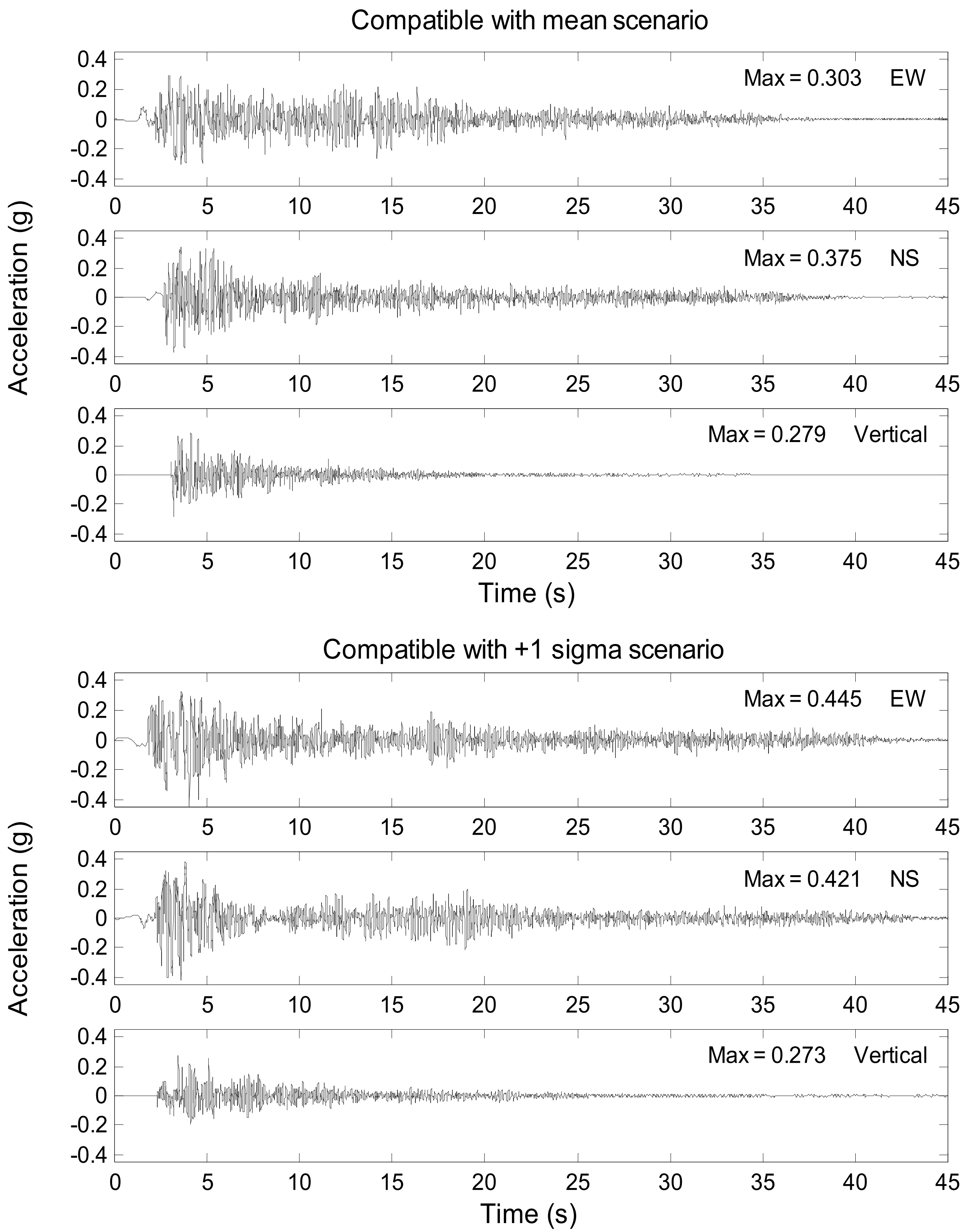


Figure 4.13 Representative acceleration time-histories for the Medical Building/PL 601 site

4.3.3 Comparison of Surface Motion Estimates at the two UCSD Sites

The horizontal surface spectral accelerations of the Thornton/Cancer Center and Medical Building/PL 601 sites are compared in Figure 4.14 for a mean scenario. There is not a significant difference between the expected strong motions at those two sites at periods above 0.17 s (frequencies below about 6 Hz).

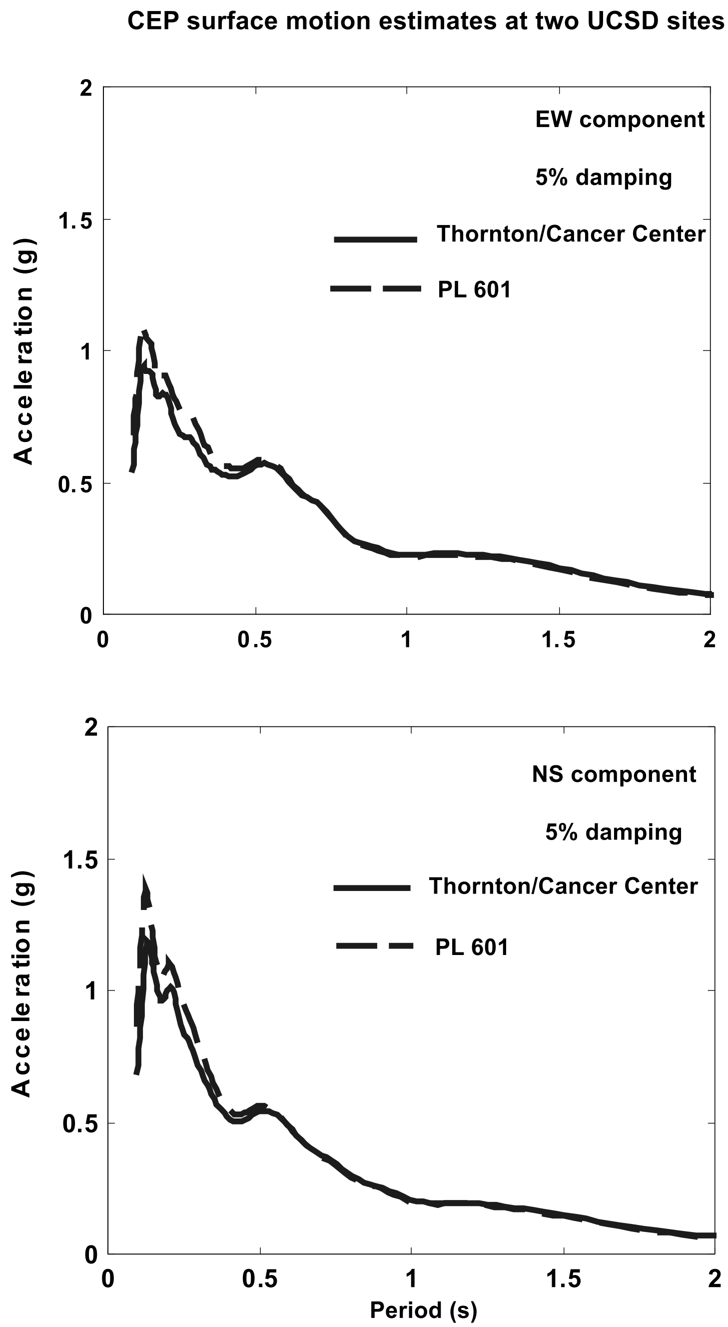


Figure 4.14 Comparison of mean horizontal acceleration response spectra for the 2 UCSD site

4.3.4 Nonlinear Behavior of UCSD Soils

The CEP estimates of surface motions use nonlinear soil dynamics models because of the presumed nonlinear response of soils to the strong motions. This assumption is corroborated by the level of shear strains expected in a M 6.9 event on the Rose Canyon fault. Figure 4.15 shows the profiles of maximum shear strain in the UCSD soil column, versus depth, for a mean and a mean + 1 sigma scenario in the EW direction at both UCSD sites. Based on the modulus degradation curves of section 4.1.2 (Figure 4.4) the shear modulus of the soils in the top 30 m at those sites may be reduced by up to 35% in a mean scenario and 50% in a + 1 sigma scenario

4.4 Overall Comparison of the CEP and State-of-the-Practice Estimates

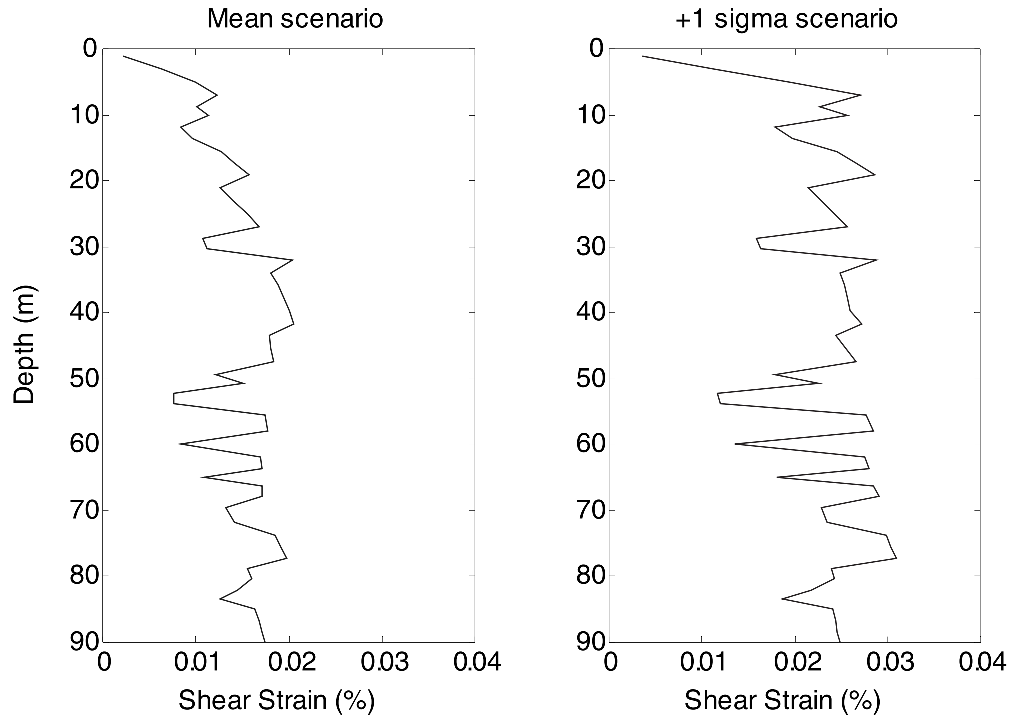
4.4.1 UBC, IBC, and PSHA for U.C. San Diego

Typically, one would obtain ground motion estimates for the UCSD sites by using other approaches. One is the 1997 Uniform Building Code (UBC 97) procedure. The outcome is shown in Figure 4.16 for 5% damping and for a Soil C site condition, based on the results of the CEP geophysical logging, and on the relevant causative fault(s) (see International Conference of Building Officials/ICBO, 1998). We also show the General Procedure Response Spectrum based on the 2000 International Building Code (ICBO, 2000).

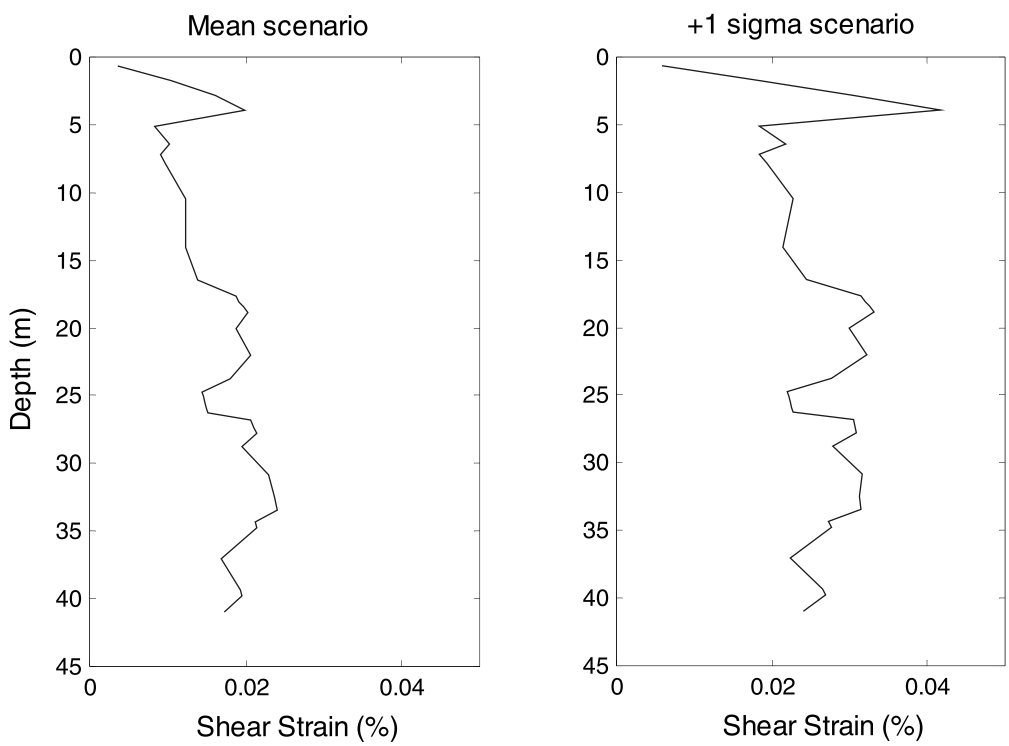
Another approach is to obtain estimates from Probabilistic Seismic Hazard Analyses (PSHA), such as those based on the research of the California Department of Mines and Geology (Petersen et al, 1996; Blake, 1999). The results are also shown on Figure 4.16 for recurrence probabilities of 10%, 5%, and 2% in 50 years (return periods of 475, 950, and 2375 years respectively).

4.4.2 Previous bases for Thornton hospital seismic retrofit

The acceleration spectra of the Maximum Credible earthquake and the Maximum Probable earthquake (DBE) used for the retrofit of Thornton hospital (Woodward-Clyde Consultants, 1989; Stedman and Dyson, Structural Engineers, 1999) are shown on Figure 4.17 These assumptions supersede the information initially provided in a study by Geocon Incorporated (1988). It is notable that these two earthquakes just about bracket the set of PSHA and code assumptions. The DBE itself is essentially consistent with a 475-year return period event.



a) Thornton hospital/Cancer Center



b) PL 601

Figure 4.15 Maximum EW shear strains in the soil column of the two UCSD sites for mean and + 1 sigma scenarios, under a M 6.9 event

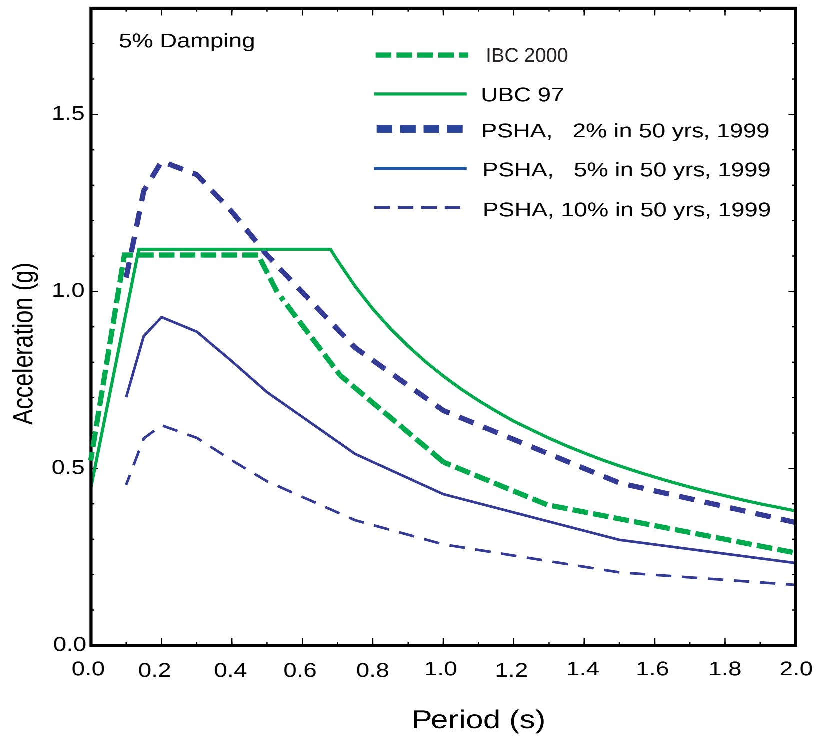


Figure 4.16 Comparison of various state-of-the-practice horizontal surface motions for UCSD

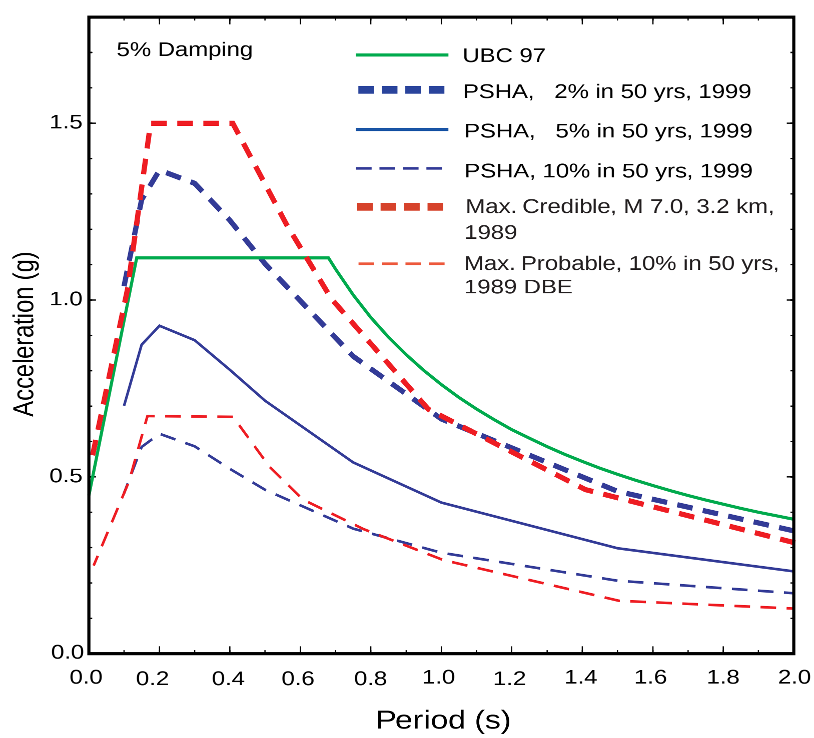


Figure 4.17 Acceleration spectra of Maximum Credible and Maximum Probable earthquakes used in the retrofit design of Thornton hospital

4.4.3 Comparison of the CEP and state-of-the-practice estimates

The CEP estimates of surface motions are then compared to those from the state-of-the-practice in Figure 4.18. Because of the similarity of estimated motions between the two UCSD sites and for both horizontal directions, we choose to show only the comparison for the EW motions at the site on the main UCSD campus (PL 601). The code-based spectra accommodate a large portion of the exposure defined by the CEP study. Also, the mean CEP is between the 475 and 950-year event PSHA spectra, and the + 1 sigma CEP is between the 950 and 2375-year event PSHA spectra.

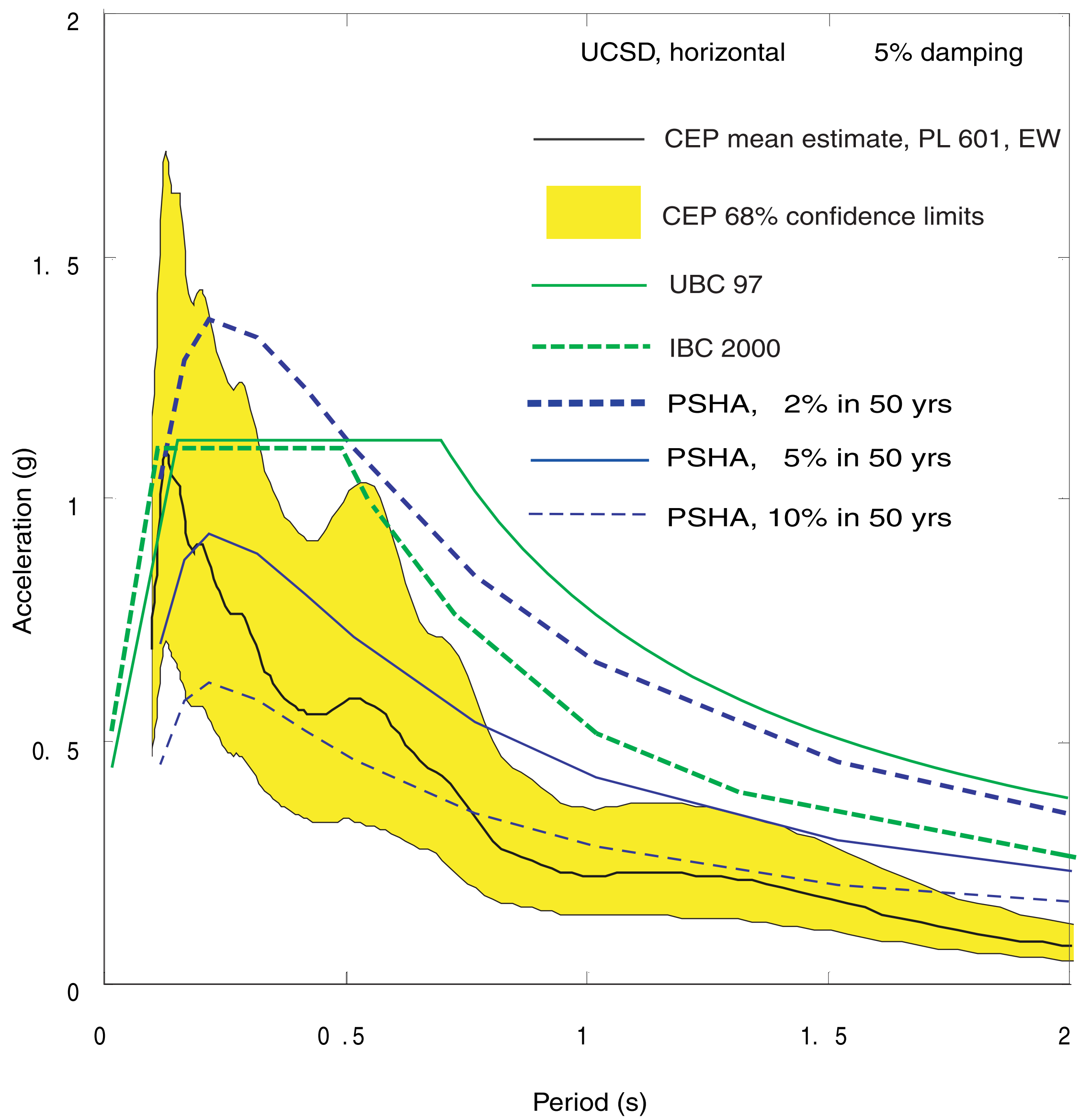


Figure 4.18 Comparison of CEP and state-of-the-practice horizontal surface motions for UCSD

4.4.4 Comparison of the CEP estimates and the Thornton Design Spectra

The CEP estimates are also compared below to the design spectra for Thornton hospital, which were shown in Figure 4.17. That comparison shows that the maximum credible event is well in excess of the + 1 sigma CEP spectrum and thus covers a very large part of all the scenarios considered in the CEP study. The maximum probable event, on the other hand, is substantially underestimating the range of estimated CEP motions.

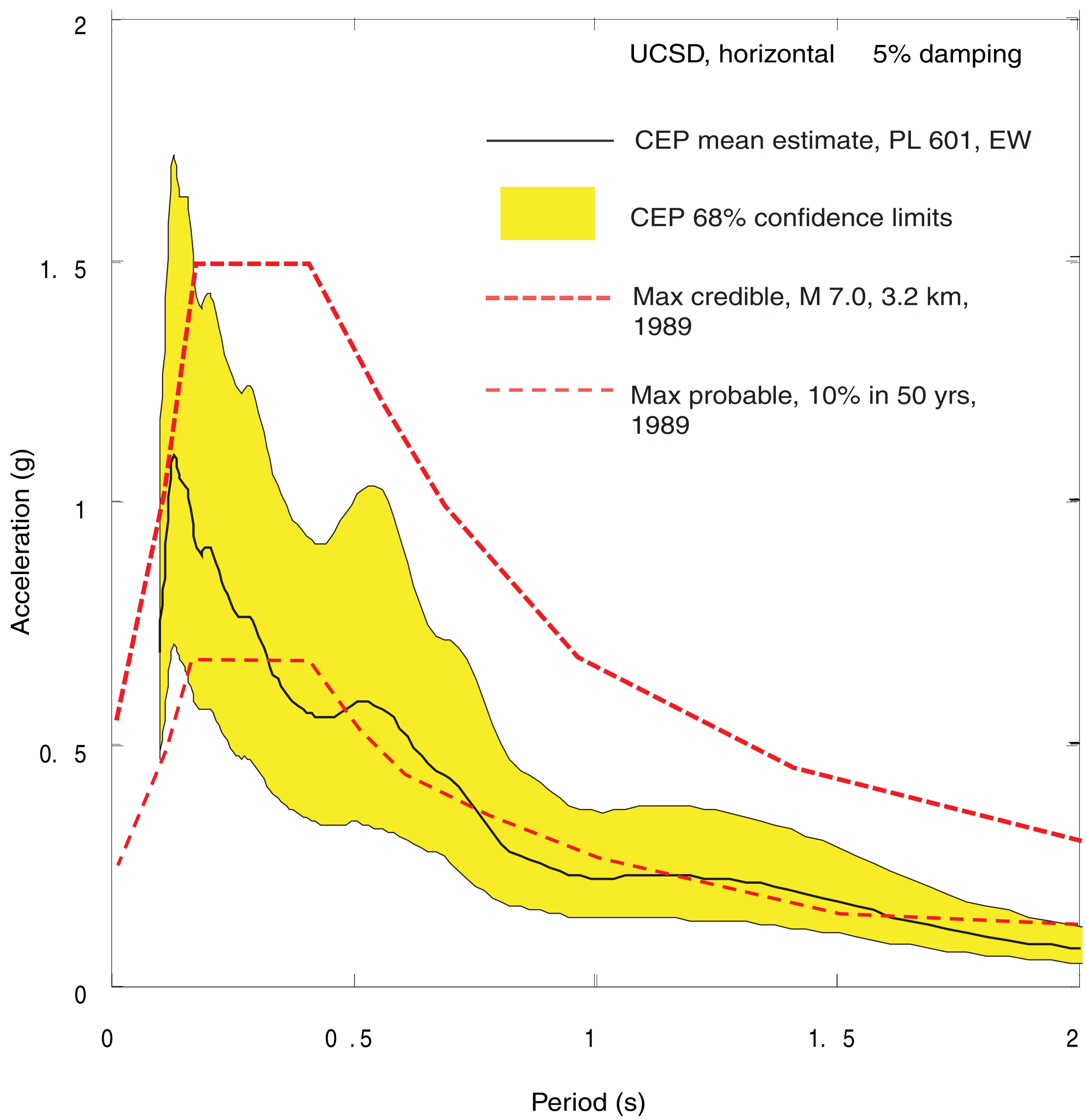


Figure 4.19 Comparison of CEP Estimates and Design Basis Spectra for Thornton Hospital

4.5 Records from Recent M ~ 7 Earthquakes in California

The CEP estimated ground motions relate to a campus that is only 4 to 5 km away from fault segments capable of producing a M 6.9 earthquake. These estimates are significantly stronger than the maximum probable event adopted for Thornton hospital. In order to "calibrate" the magnitude of the CEP-estimated accelerations we show for reference, acceleration spectra (Figure 4.20) and acceleration time-histories (Figures 4.21) corresponding to stations at somewhat comparable distances from recent events of comparable magnitude in California. These were generally strike-slip earthquakes, as that assumed for the Rose Canyon fault. Such data indicate that the CEP-estimated motions are not overestimates.

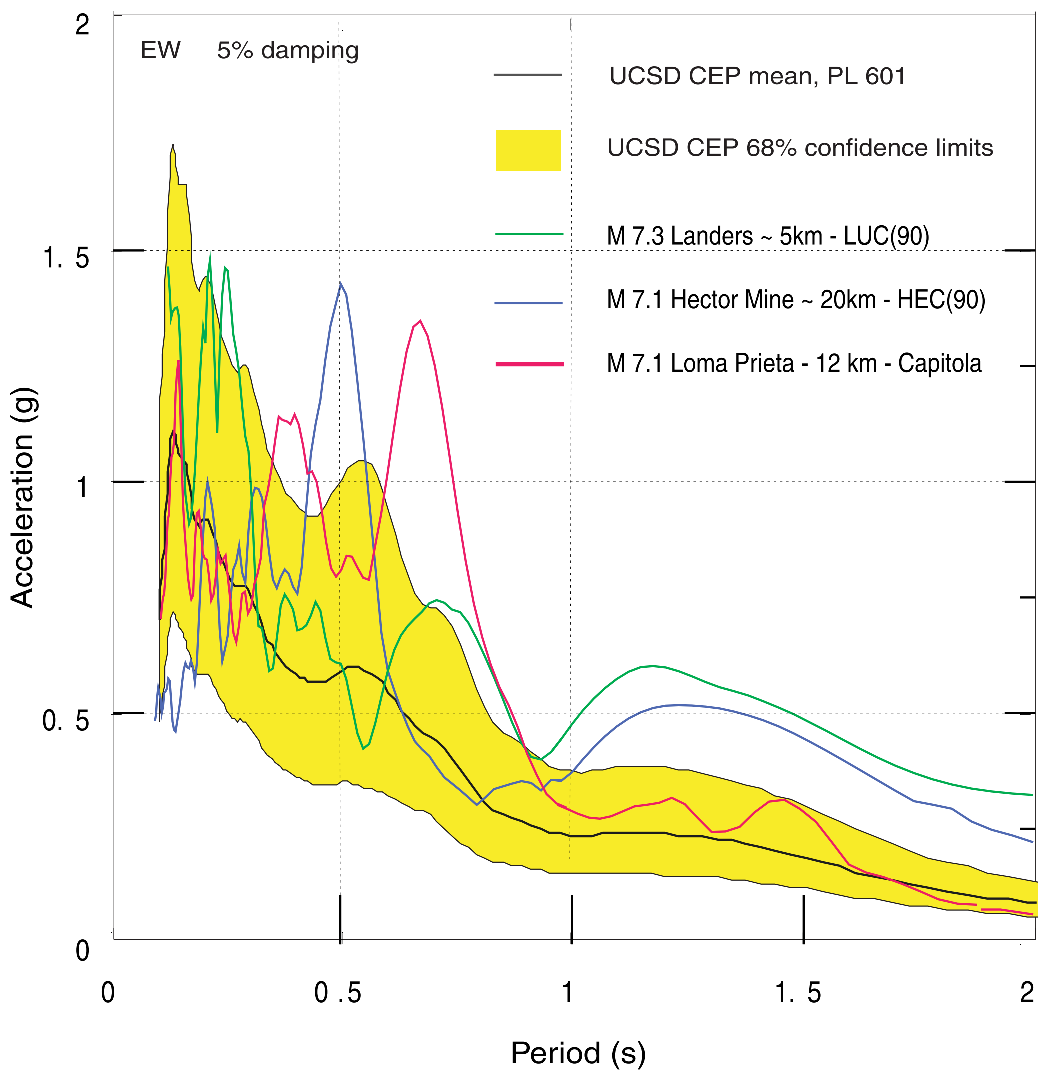


Figure 4.20 Horizontal acceleration spectra for recent M ~ 7 earthquakes in California at ranges comparable to the distance from UCSD to the Rose Canyon fault

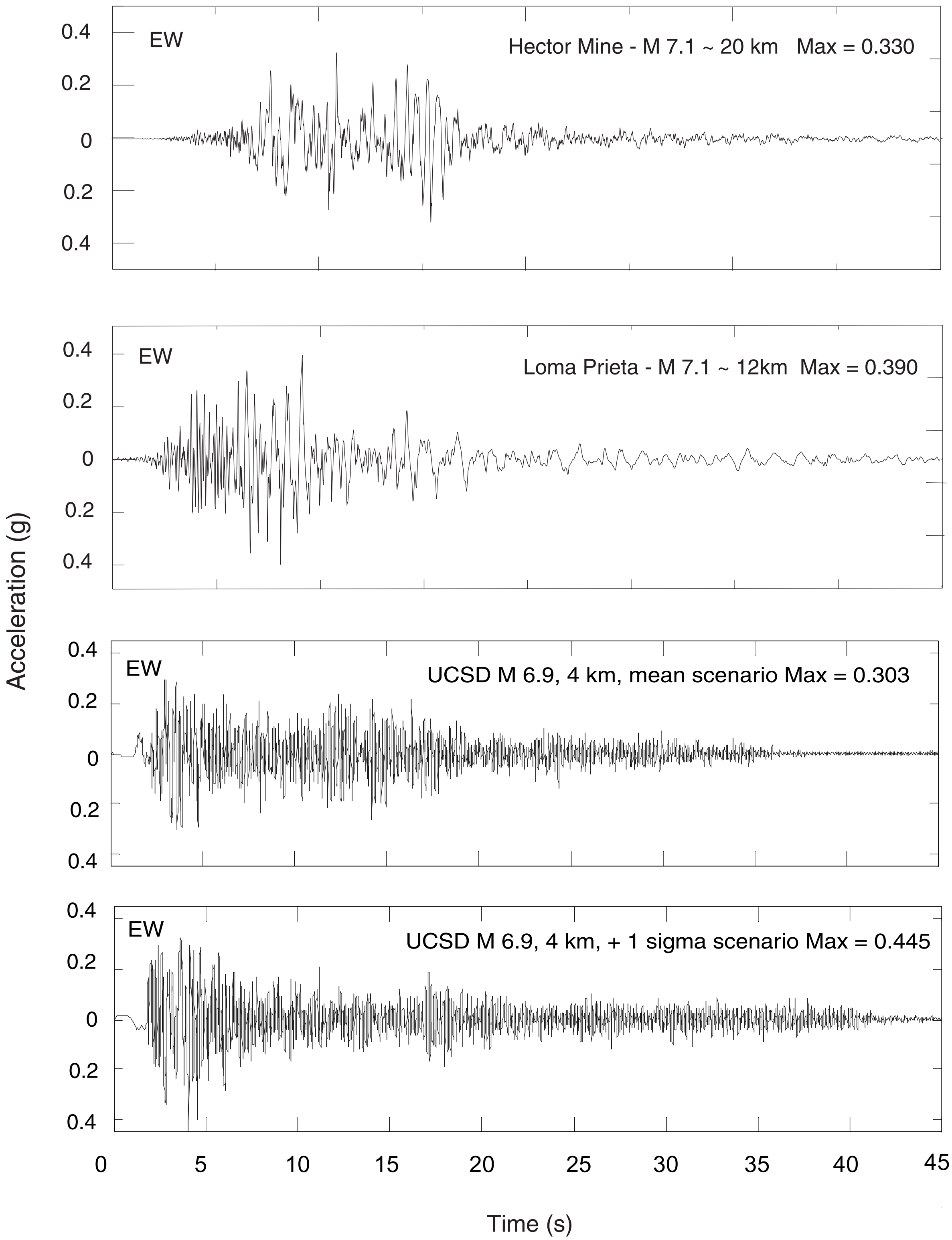


Figure 4.21 Horizontal acceleration time-histories for recent M ~ 7 earthquakes in California compared to the CEP estimates for UCSD

5.0 SUMMARY

This is the second report on the UC/CLC Campus Earthquake Program (CEP), concerning the estimation of exposure of the U.C. San Diego campus to strong earthquake motions (Phase 2 study). The first report (Phase 1), dated July 1999, covered the following topics:

- seismotectonic study of the San Diego region
- definition of causative faults threatening the UCSD campus
- geophysical and geotechnical characterization of the Thornton hospital site
- installation of the new CEP seismic station
- and, initial acquisition of earthquake data on campus.

The main results of Phase 1 are summarized in the current report.

This document describes the studies which resulted in site-specific strong motion estimates for three sites on campus: Thornton hospital, the new Cancer Center site, and Parking Lot/PL 601 site of the future Garamendi Medical Research building. The main elements of Phase 2 are:

- Concluding that a M 6.9 earthquake involving the 60-km long combined Mission Bay and Del Mar segments of the Rose Canyon fault is the main seismic hazard for the campus. Its recurrence interval is of the order of 1,800 years. The nearest distance from the surface trace of the fault segments to campus locations is 3 to 4 km.
- Creating strong motion estimates (seismic syntheses) at depth of 91 m under the Thornton hospital site, and 42 m under the PL 601 site; 300 such simulations were performed, each with the same seismic moment, but giving a broad range of motions which were analyzed for their mean and standard deviation.
- Laboratory testing, at U.C. Berkeley and U.C. Los Angeles, of soil samples obtained from drilling at the UCSD station site (Thornton hospital), to determine their response to earthquake-type loading.
- Performing nonlinear soil dynamic calculations, using the soil properties determined in-situ and in the laboratory, to calculate the surface strong motions resulting from the seismic syntheses at depth.
- Comparing these CEP-generated strong motion estimates to acceleration spectra based on the application of state-of-the-practice methods - the IBC 2000 code, the UBC 97 code, and Probabilistic Seismic Hazard Analysis (PSHA). This comparison can be used to formulate design-basis spectra for future buildings and retrofits at UCSD.

Because of the new, site-specific approach that the CEP studies represent, an extensive effort of validation is documented:

- validation of the Green's function methodology used in the seismic syntheses of strong motions at depth
- cross-comparison of the nonlinear soil models used to obtain strong motions at the surface

The ever-growing database of strong earthquake records clearly demonstrates the potential for great variability of ground motions from site to site in a given earthquake. These variations are only reflected in a coarse way in the state-of-the-practice Probabilistic Seismic Hazard Analyses, which are rather generic. Furthermore, these variations are not adequately described by the simplified design spectra of the Building codes (UBC 97, IBC 2000). These shortcomings provide a strong justification for augmenting the state-of-the-practice estimates with site-specific studies such as done by the Campus Earthquake Program.

At UCSD, the Phase 2 studies lead to the following important conclusions:

- The motions estimated at two sites on either side of Interstate 5 (Thornton hospital and PL 601) are generally comparable. These motions are expected to be representative of those that could be expected at other campus sites where the soil profile is comparable.
- However, prior to assuming the same motions for the site of a new construction project or retrofit, it is advisable to determine the shear- and compressional-wave velocities in the top 40 meters or so of depth in order to verify the similarity with the sites studied under the CEP. The preferred method for these measurements is the suspension logging described in this report. This type of investigation would add only a modest cost to standard geotechnical investigations.
- When comparing the CEP estimates to those from the state-of-the-practice, the UBC and IBC code-based spectra accommodate a large portion of the exposure defined by the CEP study. Also, the mean CEP is between the 475 and 950-year event PSHA spectra, and the + 1 sigma CEP is between the 950 and 2375-year event PSHA spectra.
- When comparing the CEP estimates to the design basis earthquakes used for Thornton hospital, that comparison shows that the maximum credible event is well in excess of the + 1 sigma CEP spectrum and thus covers a very large part of all the scenarios considered in the CEP study. The maximum probable event, on the other hand, is substantially underestimating the range of estimated CEP motions. In consequence, the seismic design-basis assumptions for UCSD should be re-examined in consultation with the campus geotechnical and structural consultants.
- A comparison of the CEP-estimated acceleration spectra and acceleration time-histories to those from recent strike-slip events in California of comparable magnitude to that assumed for the Rose Canyon fault indicates that the CEP motions are not overestimates.

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